CHAPTER 18

COASTAL ZONE

CHAPTER 18 – TABLE OF CONTENTS

18.1 INTRODUCTION

Highways that encroach upon coastal zones, including bays, estuaries and tidal basins and the shore of large lakes and reservoirs, present unique circumstances that the designer must address. Much of the discussion in the other Chapters of this *Manual* applies to these unique areas but does not address in detail the special aspects of seasonal variation and extremes of wind, wave, current and tide upon banks and shores covered in this Chapter. An overview of the hydraulics of the coastal zone can be found in Chapter 11 of the *Highway Drainage Guidelines (1)*.

18.1.1 General Definitions

BACKSHORE. The portion of the beach between the foreshore (active wave and tide zone) and the dunes or landward extend of the storm beach. The backshore is generally only acted upon by waves during storms.

BARYCENTER. The center of mass of the earth-moon system. This is the center of rotation of this two-body system.

COASTAL ZONE. The line forming the boundary between the land and water is commonly called the coastline or shoreline. The strip of land of indefinite width that extends inland to the first major change in terrain features is commonly referred to as the coast or coastal zone. The coastal zone may be several miles wide.

EMBAYMENT. An indentation in the shoreline. Open bays may be present along a shoreline where embayments are located.

FORESHORE. The portion of the beach between the backshore and the low-tide elevation. The foreshore is the active beach where wave uprush and backwash occurs over the range of the tide.

LAKESHORE. Lakeshore, like coastal zone, refers to the strip of land from a lake shoreline inland to the first major change in terrain features. Except for tidal effects, large lakes and reservoirs of 100 mi^2 or more in area have shores that require many of the same type considerations as ocean bays and estuaries. Analogous to coastal zone tides, some reservoirs are subject to frequent changes in water surface elevation due to operational practices. On the other hand, long period fluctuations in water surface are common in the Great Lakes due to hydrologic conditions, operations to control the lake levels and hydropower operations.

LITTORAL ZONE. The region that extends seaward from the coastline to just beyond the beginning of the breaking waves. Within this zone, waves and currents transport sediments. A current is generated by the incident waves within the Littoral Zone.

STRAND. A term often used to describe a long, straight section of a coast. The term is most commonly used with reference to mainland shorelines rather than barrier island shorelines.

SYZYGY. The time when the moon and the sun are aligned such that the tidal forces of these bodies act to reinforce each other. This occurs with both the new and full moon.

STORM SURGE. An increased mean sea level elevation resulting from a buildup of water along the coast caused by wind-induced shear and pressure forces.

TIDES. Periodic fluctuations in mean water level caused by the integration of gravitational and centrifugal forces between the earth, sun, moon and other astronomical bodies. The tidal variation is usually measured in multiples of the 19-yr metonic cycle.

18.1.2 Symbols and Definitions

To provide consistency within this Chapter and throughout this *Manual*, the symbols in Table 18-1 will be used. The symbols used will also be consistent with those traditionally used in the coastal zone and coastal engineering literature. If the symbol is used for more than one definition, the symbol will be defined where it is introduced.

18.1.3 Coastal Zones and Ocean Front Locations

Wave action is the erosive force affecting the reliability of highway locations along the coast. Headlands and rocks that have historically withstood the relentless pounding of tides and waves can usually be relied on to protect adjacent highway locations. The need for shore protection structures is generally limited to highway locations at the top or bottom of bluffs having a history of sloughing and along beachfronts. The corrosive effect of salt water is a major concern for hydraulic structures located along the coastline. The long-term effect on special coatings should be monitored.

Whether the beach is the foreshore of an embayment or a strand, the problems include:

- attack by waves,
- alongshore and offshore transport of beach sands,
- seasonal shifts of the shore, and
- undermining foundations of protective structures.

Wave attack on a beach is less severe than on a headland due to the gradual shoaling of the bed that trips incoming waves into a series of breakers called surf. Littoral drift of beach sands may either be an asset or a liability. If sand is plentiful, a new beach will be built in front of the embankment, reducing the depth of water at its toe and the corresponding height of the waves attacking it. If sand supply is less plentiful or subject to seasonal variations, the new beach can be induced or retained by groins under some conditions.

If sand is in limited supply, reflection from a revetment tends to degrade the beach or bed, and an allowance should be made for this type of scour when designing the revetment, both as to weight of stones and depth of foundation. Groins would be ineffective for such locations; if the groins succeeded in trapping some littoral drift, downdrift beaches would recede from a diminished supply of sediment from upcoast.

Seasonal shifts of the shoreline follow the combination in winter of:

- greater ranges of tide,
- reversal of littoral currents,
- clockwise deflection of prevailing offshore winds, and
- more frequent attack by swell.

TABLE 18-1 — Symbols and Definitions

Symbol	Definition	Units
η	Water surface elevation	ft
θ	Angle of revetment with horizontal	deg
ρ_α	Density of air	ppm
α	Angle between wave crest and shoreline	deg
γ_s	Unit weight of stone	lbs/ft^3
d_s	Depth at structure	ft
d	Water depth	ft
D	Storm duration	sec
f	Coriolis parameter/forward speed	sec^{-1}
F	Fetch (or forward speed)	ft (ft/sec)
H	Wave height	ft
H_b	Breaking wave height	ft
H_{10}	Average of highest 10% of waves	ft
H_1	Average of highest 1% of waves	ft
H_{mo}	Wave height estimated from spectrum	ft
H_t	Height of astronomic tide	ft
H_I or H_i	Incident wave height	ft
H'_o	Deepwater wave height	ft
H_o	Deepwater wave height	ft
H_s	Significant wave height	ft
K	Constant used in determining windspeed	–
K_D	Stability Coefficient in Hudson Equation	–
K_R	Refraction coefficient	–
K_S	Shoaling coefficient	–
L	Wave length	ft
P	Pressure at r from center	mb
P_o	Central pressure	mb
r	Distance from center of hurricane	ft
R	Radius to maximum winds	ft
R	Wave runup	ft
S	Specific gravity	–
S_p	Maximum storm surge height	ft
S_{tot}	Total storm surge plus astronomic tide	ft
t	Time	sec
t_d	Minimum duration of wind for fetch limited waves	sec
T	Wave period	sec
T_p	Peak period from spectrum	sec
U	Windspeed	ft/sec
U_{gr}	Gradient windspeed	ft/sec
W	Weight of stone	lb
W	Measured windspeed	ft/sec
W_A	Adjusted windspeed	ft/sec

These effects are common to the Atlantic, Pacific and Gulf coasts. The Great Lakes shorelines are not as affected by these factors. However, all of the coastlines are significantly affected by winter storms that bring large waves and storm surges. Generally, the shift is a recession, increasing the exposure of beach locations to the hazard of damage by wave action. On strands or along extensive embayments, recession at one end may occur concurrently with accretion at the other. Observations made on location should include investigation of this phenomenon. For barrier islands, the hazard may be avoided by locating the highway on the backshore facing the lagoon where possible.

Foundation conditions vary widely for beach locations. On a receding shore, good bearing may be found on soft but substantial rock underlying a thin mantel of sand. Bedding stones and even gravity walls have been founded successfully on such foundations.

Long straight beaches, spits and strands are radically different, often with softer clays or organic materials underlying the sand. Sand usually being plentiful at such locations, subsidence is a greater hazard than scour, and location should anticipate a “floating” foundation for flexible, self-adjusting types of protection.

18.1.4 Lakes and Tidal Basins

All bodies of water have wind-generated waves. The height of waves is a function of fetch, so that the larger (or longer) the lake, the higher the waves. These waves break upon reaching shoals, reducing the effects of erosion along embankments behind shallow coves and increasing the threat at headlands or along causeways in deep water. Constant rippling of tiny waves may cause severe erosion of certain soils.

Older lakes have built thick beds of precipitated silt and organic matter. Bank protection along or across such lakes must be designed to suit available foundations; it is usually more practical to use lightweight or self-adjusting types supported by soft bed materials than to excavate mud to stiffer underlying soils. The warning is especially applicable to the protection of causeway embankments.

The erosive force of wave action is a function of the fetch and, in most inland waters, is not very serious. In fresh waters, the establishment of vegetative cover can often be used to provide effective protection, but planners should not overlook the possibility of moderate erosion before the cover becomes established. Any light armor treatment should be adequate for this transitional period. Moreover, the vegetation itself may need protection until it becomes stable.

18.2 DESIGN CRITERIA

There are no generally accepted design criteria when addressing the interaction of waves on shorelines and coastal structures. Although a 100-yr flood elevation has been used extensively by State and Federal agencies concerned with coastal flooding, the transportation engineer may often be interested in the impacts of small, more frequent events.

Depending on the design situation, the designer will have to establish design criteria for relevant parameters while considering the risk and level of desired protection for the shoreline or coastal structure. For example, emergency evacuation routes should be established with the assistance of local communities and used to determine the level of protection needed for a certain highway. Some of the technical parameters for which design criteria should be established are as follows:

- tidal period;
- tide heights and elevations;
- datum;
- storm surge;
- timing of high tide relative to storm surge;
- time differences between peaks of storm surges and flood hydrographs;
- wave characteristics:
    - wind speed, direction and duration,
    - fetch,
    - wave height, and
    - wave direction;
- longshore currents and littoral transport;
- sediment particle size;
- navigational clearance:
    - vertical clearance and datum, and
    - horizontal clearance; and
- parameters affecting sensitive environmental resources.

These parameters are discussed later in more detail.

18.3 TIDES AND STORM SURGES

18.3.1 Introduction

When considering the changes that can occur along the coastal zone, one of the most important factors to consider is the changing water level. These changes are generally due to a combination of the influence of astronomic tides, storm surge and storm waves. At some locations, these changes can be extreme, with differences from the lowest elevation to the highest on the order of 25 ft. These changes can occur over a period of a day in the case of tides and over a matter of hours during storms. The significance of a fluctuating water level is apparent when one considers the role of waves in causing changes along a shoreline. During periods of relatively high water, the wave energy will be focused on a portion of the shoreline that is not normally experiencing direct wave attack. Thus, it is important to be able to estimate the extreme high- (or extreme low-) water elevations to adequately design the required level of protection or minimum elevation of a bridge over a navigable waterway.

18.3.2 Astronomic Tides

Although the actual tide at a given location is due to the influence of the moon and sun, the local atmospheric conditions and the coastline geometry, a basic explanation of these phenomena considers only the astronomical factors. Indeed, the predicted tide for most locations is solely a function of the moon and sun.

A simplified explanation of the tide-producing forces is to recognize that the center of rotation for the earth-moon system is the barycenter that is not at the center of the earth. This point of rotation is located approximately 1,068 miles beneath the earth's surface. This results in an imbalance between the gravitational forces and the centrifugal forces on the earth's surface. The net force results in a displacement of the water surface towards the moon. Of course, as the moon rotates around the earth this displacement moves across the surface of the earth. The high tide occurs where this net force is the greatest and the low tide where it is the weakest. The daily rotation of the earth results in two highs and two lows at the equator.

When an observer on the surface of the earth sees a full or a new moon, the moon is in line with the sun. The gravitational attraction of the sun adds to that of the moon for these new and full conditions. Although the mass of the sun is far greater than that of the moon, the greater distance to the sun results in a net force that is 2.5 times less than that of the moon. Nonetheless, the combination of the sun and moon result in greater tides during the full and new moons. These greater tides occur twice a lunar month (29.53 days) and are called *spring tides*. The opposite condition, when the moon and sun are at right angles, result in the smaller or *neap tides*. Thus, for most months, there will be two spring and two neap tides at any given location.

The relative role of the sun and the moon in tide generation will change during the course of both the lunar month and the solar year. These differences are in part due to the elliptical orbits of the earth around the sun and the moon around the earth.

The declination of the moon relative to the equator varies up to 23.5°. This results in a difference between the two high tides (or the two low tides) depending on one's latitude. Consider an observer standing on the surface of the earth at Position C. When the moon is in a position of high declination, the distorted body of water at Position C will be thicker than that at C′. In addition, if one examines Figure 18-1 carefully, it will be noted that there will only be one high tide and one low tide per day at C. When there is only one high tide each day, the tide is call a *diurnal tide.* At a lower latitude (e.g., A), there will still be two highs and two lows of approximately equal magnitude each day. This is called a *semidiurnal tide.* At a position such as B in Figure 18-1, there can be significant differences between the two high tides or the two low tides each day. This is referred to as a *mixed tide.* Figure 18-2 illustrates these three types of tides.

Figure 18-2 also illustrates the terminology used to describe the relative positions of the tide. For the mixed-tide condition shown in the Figure, there are two highs and two lows each day. These are called *higher high water, low water, high water* and *lower low water,* respectively. The difference between a high (or higher high) and a succeeding low (or lower low) is called the *tidal range.*

Due to the difference between the angular velocity of the moon and the angular velocity of the earth, a tidal day is approximately 24 hours and 50 minutes. The fact that the tidal day is longer than 24 hours means that, each day, the high or low water will seem to have occurred 50 minutes later than the previous day. In many engineering applications, the tidal day is approximated as 25 hours. Of course, there are other factors that will affect both the timing and the magnitude of the tide. These include the land masses that the tide must move around, the friction between the water and the ocean bottom, other astronomical bodies and the geometry of the oceans and sounds, to name a few. In addition, the local meteorological conditions, including the wind and barometric pressure, will influence the rise and fall of the ocean (or lake). These latter parameters do not follow the same predictable periodic nature of the astronomical factors and, therefore, are not included in tidal predictions.

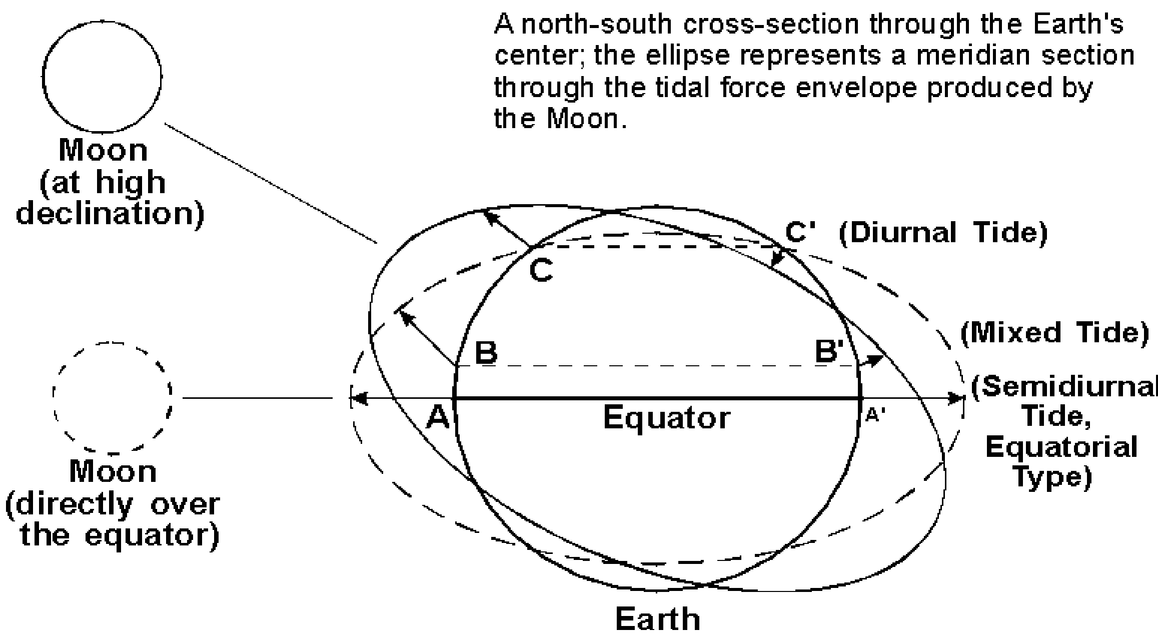


FIGURE 18-1 — The Moon's Declination Effect
(From Reference *(16)*)

NOAA provides predictions for the tides at many US and international locations. These predictions are available in published tables or via the Internet. For many years, these Tide Tables were available from the Federal government. Since 1997, the Federal government no longer publishes the tables; rather they provide the predictions to private publishers who in turn sell them to the public. An example of an internet site where tide predictions are available is http://www.co-ops.nos.noaa.gov/tp4days.html.

NOAA tide predictions are provided for both Reference Stations and secondary locations. There are approximately 50 Reference Stations. For each Reference Station, the Tide Tables provide the time of each high and low water for each day of the year. The water elevations are referenced to the local datum (often mean lower low water). Table 18-2 is an example of the predictions of *Times and Heights of High and Low Water* for Charleston, SC, reproduced from the NOAA web site.

If one is interested in a prediction for a site other than a Reference Station, then NOAA provides a second set of predictions that will correct a secondary station back to the Reference Station. Table 18-3 illustrates the corrections available for some secondary stations in South Carolina. The following explains the use of Table 18-3:

Time Differences — The columns headed "Time Differences" in which the hours and minutes to be added or subtracted from the time of high or low tide of the Reference Stations are used to determine the time of high and low tide at any station listed in this Table. A plus sign (+) indicates that the tide at the subordinate station occurs later than at the Reference Station and the difference should be added; a minus sign (-) indicates that it is earlier and should be subtracted.

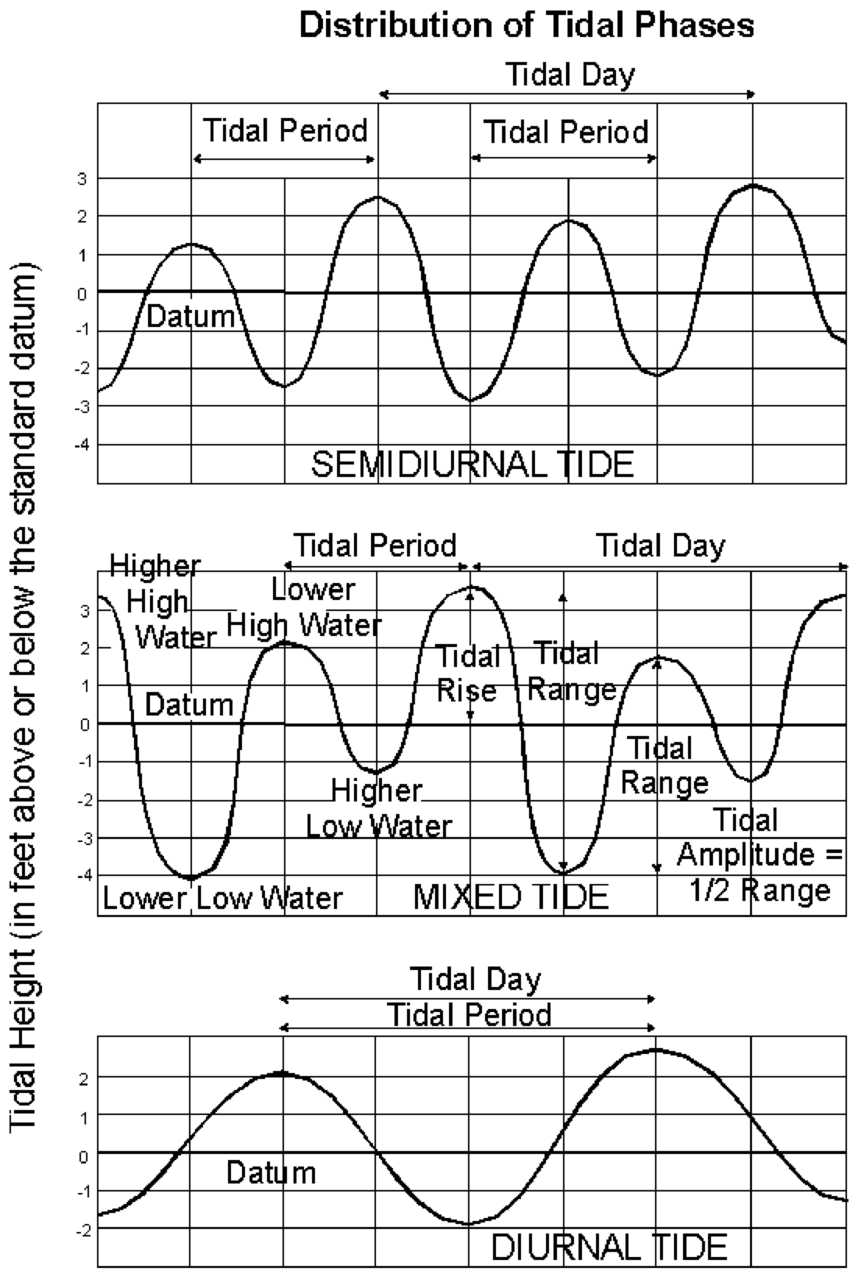


FIGURE 18-2 — The Principal Types of Tides
(from Reference *(16)*)

TABLE 18-2 — Tide Predictions (High and Low Waters) January 2001, Charleston Harbor, South Carolina

Day		Time	Ht. (ft)	Time	Ht. (ft)	Time	Ht. (ft)	Time	Ht. (ft)
1	M	550am L	0.7	1203pm H	4.8	633pm L	0.6		
2	Tu	1230am H	4.5	645am L	0.8	1252pm H	4.7	723pm L	0.5
3	W	127am H	4.7	747am L	0.9	148pm H	4.6	818pm L	0.4
4	Th	228am H	4.9	853am L	0.8	248pm H	4.5	915pm L	0.2
5	F	329am H	5.2	958am L	0.6	351pm H	4.6	1013pm L	-0.1
6	Sa	430am H	5.6	1100am L	0.3	452pm H	4.7	1110pm L	-0.4
7	Su	529am H	6.0	1157am L	0.0	551pm H	4.9		
8	M	1204am L	-0.7	626am H	6.3	1251pm L	-0.3	649pm H	5.1
9	Tu	1258am L	-1.0	721am H	6.5	143pm L	-0.5	744pm H	5.2
10	W	150am L	-1.1	814am H	6.7	234pm L	-0.7	839pm H	5.3
11	Th	242am L	-1.1	907am H	6.6	325pm L	-0.7	934pm H	5.4
12	F	335am L	-1.0	1000am H	6.5	415pm L	-0.7	1029pm H	5.4
13	Sa	428am L	-0.8	1052am H	6.2	506pm L	-0.6	1125pm H	5.4
14	Su	523am L	-0.5	1145am H	5.8	558pm L	-0.4		
15	M	1222am H	5.3	621am L	-0.2	1239pm H	5.4	652pm L	-0.2
16	Tu	120am H	5.3	721am L	0.1	134pm H	5.0	747pm L	-0.1
17	W	219am H	5.2	822am L	0.3	231pm H	4.7	842pm L	0.1
18	Th	317am H	5.2	923am L	0.4	327pm H	4.5	937pm L	0.1
19	F	413am H	5.3	1020am L	0.4	421pm H	4.4	1029pm L	0.1
20	Sa	505am H	5.4	1112am L	0.4	513pm H	4.4	1118pm L	0.0
21	Su	554am H	5.5	1200pm L	0.3	600pm H	4.5		
22	M	1204am L	-0.1	639am H	5.5	1244pm L	0.2	644pm H	4.5
23	Tu	1247am L	-0.2	720am H	5.5	126pm L	0.1	725pm H	4.6
24	W	128am L	-0.2	759am H	5.5	205pm L	0.1	803pm H	4.6
25	Th	207am L	-0.2	835am H	5.5	243pm L	0.1	838pm H	4.6
26	F	244am L	-0.1	909am H	5.4	320pm L	0.1	913pm H	4.6
27	Sa	321am L	0.0	941am H	5.2	356pm L	0.2	947pm H	4.6
28	Su	358am L	0.1	1012am H	5.1	432pm L	0.2	1023pm H	4.7
29	M	437am L	0.3	1046am H	4.9	510pm L	0.3	1104pm H	4.7
30	Tu	521am L	0.4	1124am H	4.7	551pm L	0.3	1151pm H	4.8
31	W	612am L	0.6	1210pm H	4.6	639pm L	0.3		

TABLE 18-3 — Tidal Differences (Secondary Stations in SC)

	Time Difference		Height Difference Ratios		Reference Station
Stono River Locations (upstream):					
Snake Island	+0 01	-0 12	*1.01	*1.05	Charleston
Abbapoola Creek entrance	+0 17	+0 02	*1.01	*0.95	Charleston
Elliott Cut entrance	+0 49	+0 52	*0.99	*1.21	Charleston
Pennys Creek, west entrance	+1 23	+1 21	*1.03	*1.42	Charleston
Sandblasters, Pennys Creek	+1 30	+1 19	*1.03	*1.53	Charleston
Limehouse Bridge	+1 43	+1 34	*1.08	*1.32	Charleston
Church Flats	+1 52	+1 14	*1.22	*1.26	Charleston
Kiaway River Bridge	+0 14	+0 06	*1.07	*0.95	Charleston

Source: *NOAA* – http://co-ops.nos.noaa.gov/tp4days.html.

To obtain the tide at a subordinate station on any date, apply the difference to the tide at the Reference Station for that same date. In some cases, however, to obtain an AM tide, it may be necessary to use the preceding day's PM tide at the Reference Station or, to obtain a PM tide, it may be necessary to use the following day's AM tide. For example, if a high tide at a Reference Station occurs at 0200 on July 17, and the tide at the subordinate station occurs 5 hours earlier, the high tide at the subordinate station will occur at 900 PM on July 16. For the second case, if the high water at a Reference Station occurs at 1000 PM, and the tide at the subordinate station occurs 3 hours later, then high tide will occur at 100 AM on July 3 at the subordinate station.

The results obtained by application of the time differences will be in local time for the subordinate station. The necessary allowances for the change in date when crossing the International Dateline or for different time zones have been included in the time differences listed.

Height Differences — Excluding the effect of wind and barometric pressure, the height of the tide, referred to as the datum of nautical charts, is obtained by means of the height difference or ratios. A plus sign (+) indicates that the difference should be added to the height at the Reference Station, and a minus sign (-) indicates that it should be subtracted. For most stations, use of a predicted height difference would give unsatisfactory predictions. In such cases, they have been omitted and one or two ratios, indicated by an asterisk (*), are given. To obtain the height of tide at the subordinate station in these cases, multiply the height of tide at the Reference Station by the ratio listed. The result is normally rounded to the nearest 0.1 ft.

For some subordinate stations, there is given (in parentheses) a ratio and a correction. In those instances, each predicted high and low water at the Reference Station should be first multiplied by the ratio, and then the correction is added or subtracted from each product.

There are significant differences in mean and spring tidal ranges along the United States coast. Table 18-4 illustrates some of these differences. These differences are due to the combination of the geometry and bathymetry of the coastline at each of these stations. It is interesting to note that there is not a uniform decrease in range as one moves from Maine to Florida. In fact, some of the largest tides on the US East Coast are in Georgia. Table 18-4 also illustrates the difference between the mean and the spring tide. In a conservative engineering analysis, one often uses the spring range if one is interested in considering maximum tidal flows.

TABLE 18-4 — Mean Tidal Ranges, ft

Station	Mean	Spring
Calais, ME	20	23
Chatham, MA	7	8
Cape May, NJ	4	5
Charleston, SC	5	6
Savannah, GA	7	8
Key West, FL	1	2
Port Isabel, TX	1	1
Point Loma, CA	4	5
Puget Sound, WA	11	15

Source: Reference *(22)*.

An alternative to the NOAA web site for prediction of astronomic tides is the free software *X-TIDE* that is available at http://www.wxtide32.com/. This software provides predictions at most of the same locations as does the NOAA web site, and it automatically incorporates the height and time difference.

18.3.3 Tidal Datums

A vertical datum is a reference level for elevations. The elevations used on most topographic maps, bridge plans, floodplain maps, etc., are referenced to a fixed vertical datum. A fixed datum is a reference level plane that has a constant elevation over a large geographical area. Hydraulic analysis always requires reference to a fixed vertical datum.

In the United States, the most commonly used fixed datums are those established by the National Geodetic Survey (NGS). In some locations, however, local fixed datums are used. Local datums are established by the State, city or county and are independent of the standard NGS datums.

A tidal datum is a reference level that applies at one geographic point in a tidal water body and is commonly established by the National Ocean Survey (NOS of NOAA). Much of the data available for the analysis of tidal waterways refers to tidal datums rather than fixed datums. Most bathymetric data, navigational charts and tide-level observations, for instance, present elevations above tidal datums. Tidal datums are defined on the basis of mean tide fluctuations. For more information, see http://co-ops.nos.noaa.gov/data_res.html. Other benchmarks may be available from State agencies.

Several terms related to tide levels are defined in Section 18.1.1. To explain tidal datums, some of these terms are illustrated again in Figure 18-3. Usually, the reference level for a tidal datum is mean low water (MLW) or mean lower low water (MLLW). Occasionally, mean sea level (MSL) is used as a reference datum. Section 18.3.2 includes a detailed discussion of these difference tidal terms. In general, a “mean” tide refers to an average over one tidal epoch or 18.6 yrs. For example, the mean lower low water is the 18.6-yr average of all the lower low waters for the location in question. Similarly, mean high water is the average of all the high (and higher high) water levels over the 18.6-yr period.

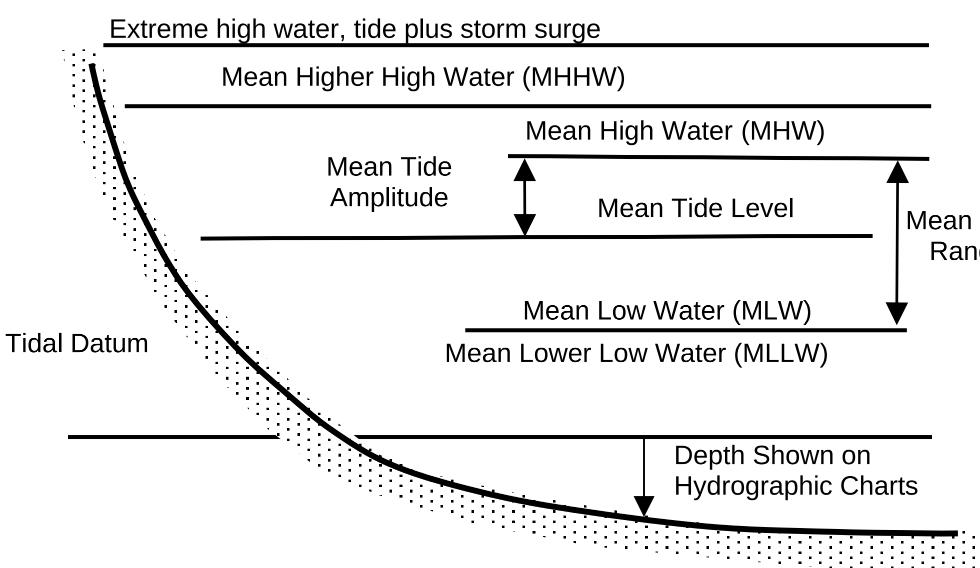


FIGURE 18-3 — Definition Sketch for Tide Level Terminology

The relationship between a tidal datum and a fixed datum can vary widely within a single estuary or bay system because the mean tide range is variable. Moreover, a tidal datum at a given point changes with time. Mean tide levels are established for a complete tidal epoch, a period of approximately 19 yrs (see Section 18.3.2). Because of relative ocean level rise, the values usually change from one tidal epoch to the next.

Confusion often arises when MSL is referenced as a vertical datum. Mean tide level (MTL) is the midway point between MLW and MHW. MSL is not necessarily synonymous with MTL. Often, when a bridge drawing or other document refers to MSL as its vertical datum, it is actually referring to a fixed datum. When the datum is named as MSL, the user of the data must clarify whether the reference is equivalent to MTL for the current tidal epoch. If it is found that MSL actually refers to a fixed datum, the user must determine the relationship between that fixed datum and the datum used in the study at hand.

The fixed datums used most commonly in the United States are the National Geodetic Vertical Datum of 1929 (NGVD29) and the North American Vertical Datum of 1988 (NAVD88). These reference surfaces were established by NGS. High-accuracy benchmarks referenced to these datums have been set by NGS throughout the nation.

NGVD29 was called the Sea Level Datum of 1929 until the name was changed in 1973. Many older plan sets, maps and documents refer to a datum of Sea Level or Mean Sea Level that is actually equivalent to NGVD29. One must not assume this equivalency, however, without verification. NGS accomplished the establishment of NGVD29 by connecting the major vertical benchmark networks in the nation to 26 tidal benchmarks along the Atlantic, Gulf and Pacific Coasts.

NAVD88 was established by NGS in 1991. It is considered to be a significant technical improvement over NGVD29 in accuracy and applicability over very large areas. All new NGS benchmarks are established in NAVD88. The difference between NGVD29 and NAVD88 is variable, depending on region.

Prior to the Internet, the conversion of elevations from a tidal datum to a fixed datum was usually a difficult process. Now, however, the required information is readily available on the Internet (e.g., http://www.ngs.noaa.gov/cgi-bin/ngs_opsd.prl). The relationship between NGVD29 and NAVD88 varies geographically. If a conversion between these two fixed datums is necessary, the locally appropriate adjustment should be verified.

The USACE CorpsCon program is an excellent program for converting large data sets from one fixed vertical datum to another (http://crunch.tec.army.mil/software/corpscon/corpscon.html). It includes a database of conversion constants for the entire nation. CorpsCon also converts horizontal coordinates between various systems, such as Latitude-Longitude to Universal Transverse Mercator (UTM) or state-plane, etc. There are limitations with CorpsCon because it does not convert to NGS Geoid99 or WGS 84. These conversions can also be accomplished in commercially available software packages such as SMS (Surface Water Modeling System). SMS will be discussed in more detail in Section 18.8.3.

Bridges in coastal zones are typically navigable, and special design considerations are needed for this purpose. It is necessary for the designer to consult with USCG to establish requirements for navigational clearance to allow for floating vessels to pass under the roadway. It is important to identify both vertical and horizontal minimum clearances at design conditions that should be incorporated into the design. Because bridges in coastal zones are also subject to scour and erosion during different storm conditions and combinations such as hurricanes superimposed with runoff events, due consideration should be given to such complexities in design.

Small highway drainage systems such as culverts and storm drains are also affected by coastal zone processes. Hydraulic performance of culverts that drain runoff events from upstream high ground into the coastal zone is affected by the tidal action in many ways. The most obvious impact in such cases is the change in tailwater on the headwater. However, careful consideration to sediment deposition or scour possibilities inside and around the culvert should also be given. The structural integrity of the culverts and storm drains exposed to extended stagnant water levels should also be considered during design. Special consideration and protection is also warranted for the backfill material for the coastal small drainage structures.

Coastal zones serve as habitat for sensitive aquatic resources. Design of transportation structures in coastal zones must incorporate protection of such resources. Because each natural resource has a different requirement to survive, the designer must work with specialists to develop design criteria that lead to balanced designs.

Example Problem

A navigational chart shows a depth of 35 ft in the channel as it passes under the bridge. Determine the elevation of the channel bed in NAVD88 and MSL. The local benchmark has the following information:

MHHW	7.89 ft
MHW	7.53 ft
NAVD88	4.34 ft
MSL	3.90 ft
NGVD29	3.27 ft
MLW	0.26 ft
MLLW	0.00 ft

The elevation of the bottom of the channel in North American Vertical Datum is -35 – 4.34 = -39.34 ft NAVD88. For reference to mean sea level, the elevation is -35 – 3.90 = -38.90 ft MSL. Determine next the maximum clearance for a vessel to pass beneath the bridge that has a bottom chord elevation of 72.00 ft NAVD88. It would be important to use the worse case in this situation because there is no way to know when the vessel would be traveling the waterway. So, the use of MHHW would give a minimum clearance of 72.0 – (7.89 – 4.34) = 68.45 ft. Using the NGVD29 reference datum would have given more clearance than actually exists.

18.4 DYNAMIC BEACH PROCESSES

18.4.1 Introduction

The beach and near-shore zone of a coast is the region where the forces of the sea react against the land. The physical system within this region is composed primarily of the motion of the sea, which supplies energy to the system, and the shore, which absorbs this energy. Because the shoreline is the intersection of the air, land and water, the physical interactions that occur in this region are unique, very complex and difficult to fully understand. Although there have been significant advances in understanding beach processes in recent years, the ability to predict changes is still limited.

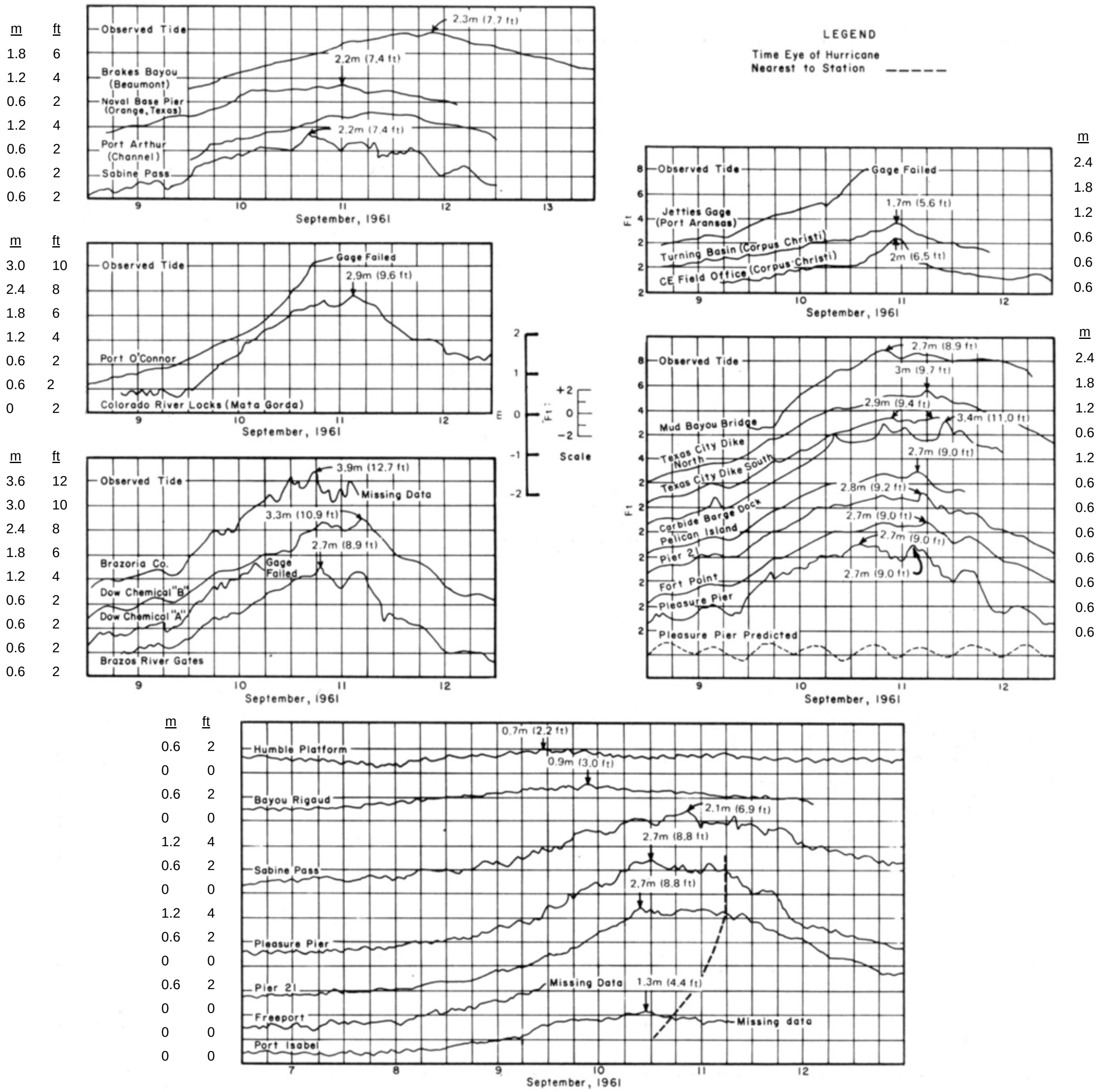
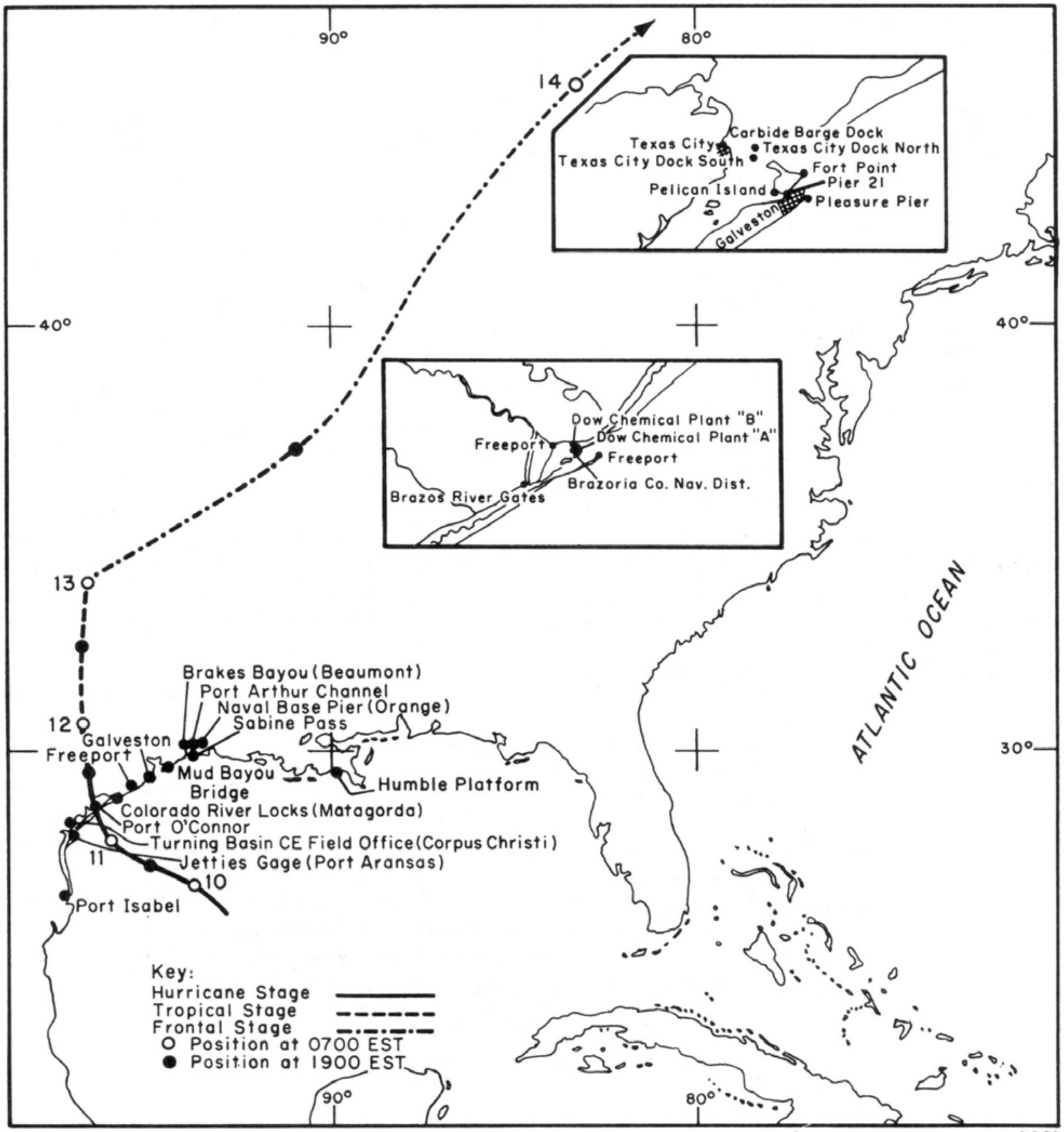


FIGURE 18-6 — Storm Surge Elevations for Hurricane Carla
(Reference *(22)*)



(from Harris, 1963)

FIGURE 18-7 — Storm Track for Hurricane Carla, 7-12 September 1961 (Reference *(22)*)

On coasts where the shoreline is unconsolidated sediment (e.g., a clay, sand, silt), the energy from the waves, wind and tide can cause rapid change in the shape and dimensions of the shoreline. Waves are the most significant factor to cause shoreline change. As waves move from offshore to the beach, they will often break, reform and break again. The process of breaking results in a portion of the wave energy being dissipated. Additional energy is dissipated on the beach with the resultant transport of the beach sediment.

Figures 18-8 and 18-9 illustrate the principal features of the beach and nearshore wave environment, or the littoral zone. The offshore region lies beyond the zone of wave breaking. On many sandy coasts, the landward end of this region is characterized by the presence of a longshore bar. The inshore region extends from the bar (or bars) across the surf zone to the position of the tidal low-water line. The foreshore extends from the low-water line to the upper limit of swash and the beginning of the beach backshore. On beaches where dunes are present, the seaward toe of the dune marks the end of the backshore. If dunes are not present on the beach, the landward limit of the beach backshore is generally considered to be the upper limit of storm wave impacts. Other important features illustrated in these Figures include the berm and the trough (just inshore of the alongshore bar).

The widths of the breaker and surf zones shown in Figure 18-9 change with wave conditions. During storms, when the waves are relatively large, these zones extend further offshore as the waves break in deeper water. Similarly, the swash zone will also be larger (and penetrate further landward) during storm conditions. A complete discussion of the nature of waves and sediment transport can be found in Reference *(19)*.

18.4.2 Normal Conditions

As a wave moves toward the shore, it will break when the wave height is equal to about three-quarters of the water depth. The actual depth at breaking is a function of the beach slope and the wave length and period. Breakers are classified as four types — plunging, spilling, surging and collapsing. Plunging breakers have distinct curls, spilling breakers break more gradually and have characteristic white water, and surging breakers begin to form a plunging face, but reach the beach before this face is formed. Collapsing breakers are a transition category between plunging and surging.

The form of breakers is controlled by wave steepness and nearshore bottom slope. Breaking results in a dissipation of wave energy by the generation of turbulence in the water and by the suspension and transport of sediment. The broken wave forms a bore that moves across the surf zone to the beach where it forms the wave uprush and backwash in the swash zone. The formation of the bar is directly related to the sediment-transport characteristics of the breaking waves. The dimensions and locations of the various wave zones are functions of the wave characteristics and the stage of the tide. Regions with relatively large tide ranges will have wider limits to the positions of these zones.

18.4.3 Storm Conditions

The high winds associated with storms generate large waves. In open water, the actual size and period of the waves are a result of a combination of the size of the storm (fetch), the length of time the storm winds have been blowing across the fetch (duration) and, of course, the magnitude of the wind itself. In enclosed bodies of water, such as bays and estuaries, the shape of the shoreline and the depth of the water also affect the wave conditions. A method is presented in Section 18.6 to estimate the wave conditions. As the storm waves move to the coast, they are modified by the presence of the shallow water and, when they reach their limiting depth, they break. These breakers, and the associated energy dissipated, are greater than during normal conditions and, therefore, there is more energy available to erode the shoreline. These changes often include the movement of the bar offshore, the recession of the beach and, in extreme storms, the erosion of the dune. Because the storm conditions may also include the presence of a storm surge, the portion of the beach profile exposed to wave attack is greater than during normal non-storm conditions.

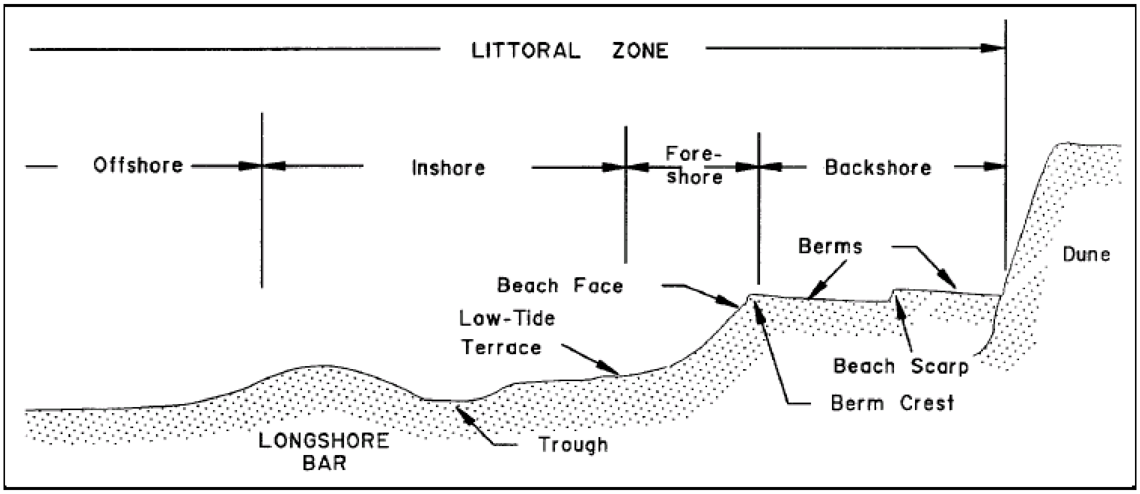


FIGURE 18-8 — Beach Profile Terminology *(19)*

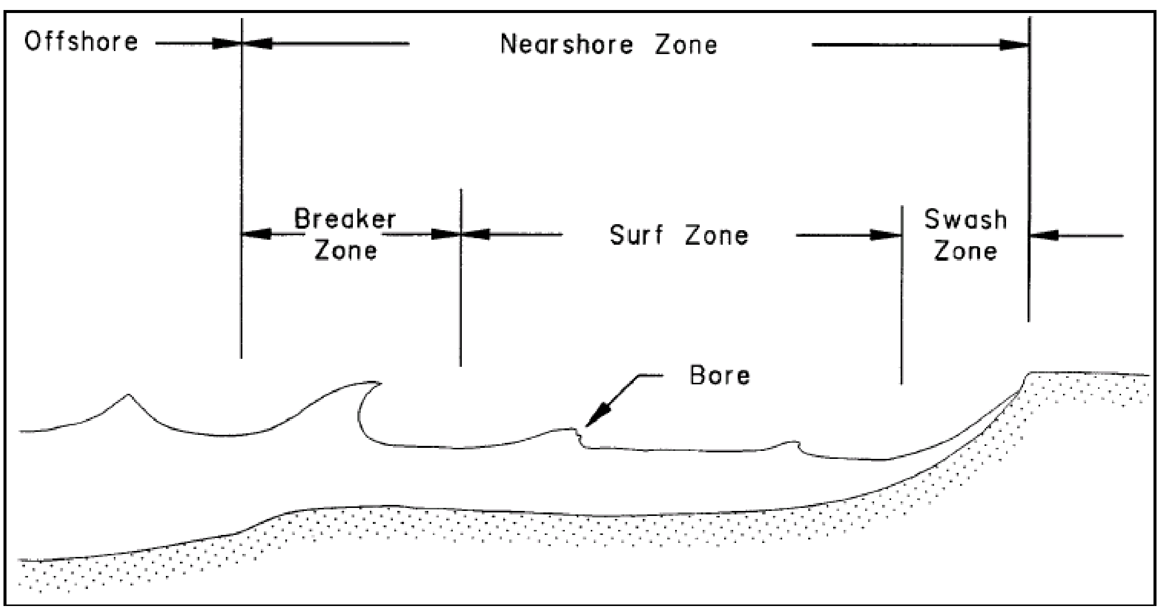


FIGURE 18-9 — Nearshore Wave Processes Terminology *(19)*

Figure 18-10 illustrates the changes that are likely to occur on a beach as a result of a storm. As the waves and surge increase, sediment is moved offshore as the bar migrates to deeper water. The bar may in fact grow large enough to cause the storm waves to break further offshore, thereby reducing the wave energy in the breaker zone. This process of bar migration offshore can be thought of as a process by which the shoreline is protecting itself from further erosion by the storm waves. Figure 18-10, Profile B, illustrates this mechanism.

The beach berm is naturally built by the waves during periods of relatively low wave energy and sediment accretion. The berm elevation approximates the highest elevation reached by normal waves. When storm waves erode the berm and transport the sediment offshore, the protective value of the berm is reduced, and large waves can penetrate further landward across the beach backshore. The width of the berm at the time of a storm is thus an important factor in the amount of dune and upland damage a storm can inflict.

During severe storms, such as hurricanes (or large northeasters), the higher water levels resulting from storm surge may lead to dune erosion. It is not unusual for 65-ft to 100-ft wide dunes to disappear in a few hours. This dune erosion will be greater when the period of maximum storm surge coincides with a high astronomic tide (see Figure 18-10, Profile C).

After the storm has passed and the waves return to normal size and period, the beach goes through a period of recovery. Material is transported from the bar and nearshore profile back to the beach above mean water level. The berm builds out and, when the sediment dries, is transported by the wind to rebuild the dune. This mechanism of beach rebuilding is illustrated in Figure 18-10, Profile D.

During very large storms, the combination of the surge and large waves may succeed in completely overtopping the dunes causing extensive coastal flooding. When this occurs, the water transports beach and dune sediments landward in a process referred to as overwash. In some cases, on barrier islands, the overwash may transport sediment completely across the island and deposit the material in the estuary (sound or bay). This transport of material out of the littoral zone represents a net loss of material from the beach and nearshore. In rare cases, the overwash and storm flooding (from both the ocean and the estuary) may erode enough sediment to cut an inlet across the island. Such an inlet may close within a matter of weeks or months or, in extreme cases, become a new feature of the barrier island.

18.4.4 Beach and Dune Recovery

Following a storm there is a return to more normal conditions that are characterized by low wave heights and longer periods than during storms. These waves that are not generated by the local winds along the coast are termed *swell*. As noted above, these waves tend to transport material back to the shoreline, moving the bar shoreward and rebuilding the berm. Often, the rebuilding of the beach is incomplete, because there is a net loss of material from the system as material is transported far offshore or along the beach. This latter transport is referred to as *longshore transport*.

On some shorelines, there is a characteristic seasonal change in the shape of the beach. During the winter months, with relatively frequent storms, the beach is cut back, so it appears to be relatively narrow and flat. During the summer months, the beach rebuilds, the berm widens and the foreshore returns to the characteristic summer profile.

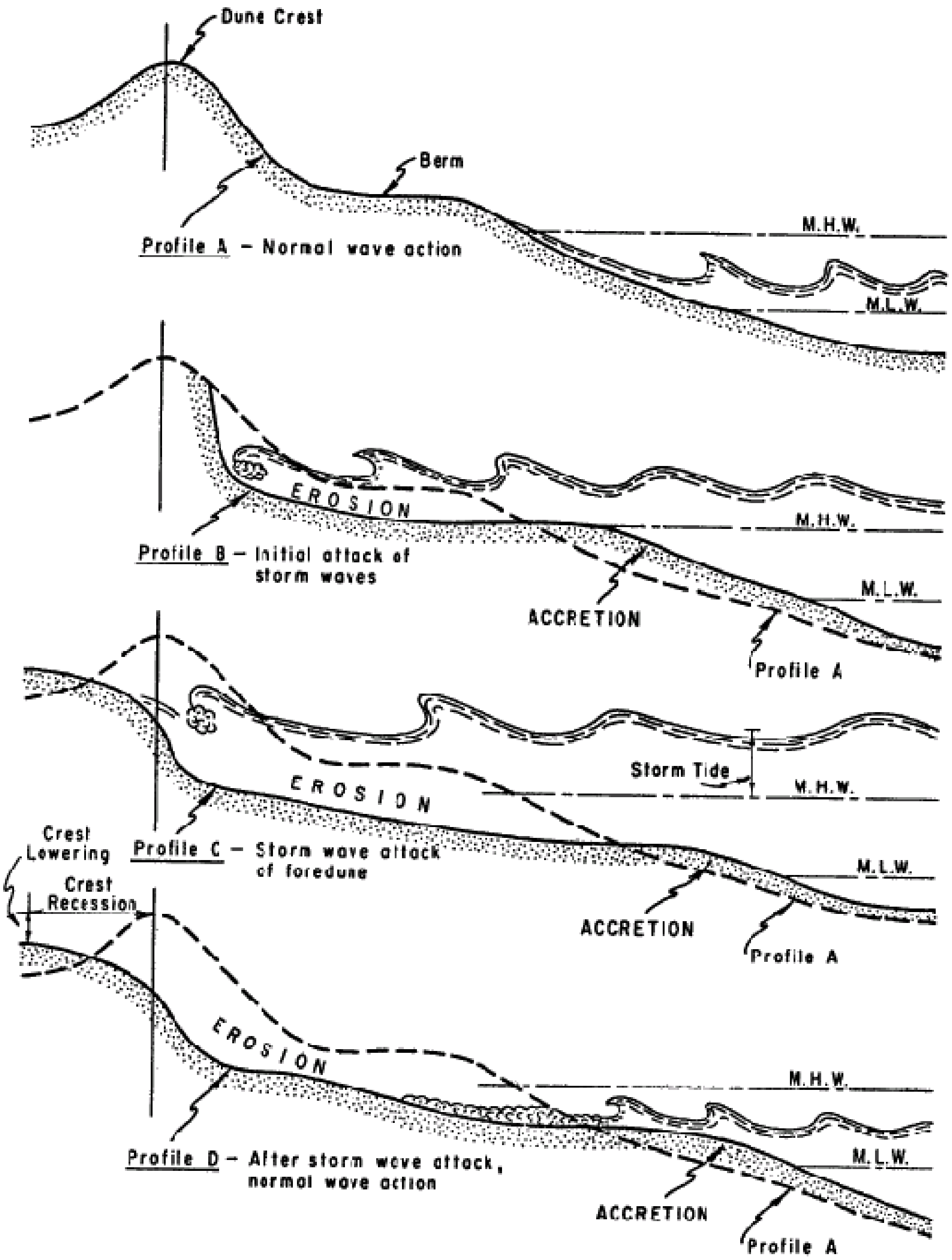


FIGURE 18-10 — Schematic Diagram of Storm Wave Attack on Beach and Dune (from Reference *(22)*)

18.4.5

Modeling of Shoreline Change

There is a growing list of computer models for predicting shoreline change. These models can be separated into two general types — onshore/offshore models (cross shore) and longshore models. The models include both highly empirical models based on a combination of field and laboratory data and theoretical models based upon the underlining hydrodynamic equations. An up-to-date summary of these models can be found in Dean and Dalrymple *(8)*. Alternatively, one can find reference to the models developed by USACE at the Internet site for the Coastal & Hydraulics Laboratory (http://chl.wes.army.mil/).

One of the more frequently used models from the Coastal & Hydraulics Laboratory (CHL) for cross-shore sediment transport is SBEACH. SBEACH (Storm-Induced Beach Change) is one of a handful of models that are currently being used by coastal engineers to determine shoreline changes due to storms. SBEACH divides the nearshore into four distinct zones — swash, broken wave, breaker transition and pre-breaking. In each zone, a set of empirical equations are used to model the transport of sediment (sand) across the zone under the influence of the combination of the astronomical tide, wind waves and storm surge. Therefore, the inputs to the model include a two-dimensional description of the shoreline extending from offshore to the landward limit of interest, a characterization of the average sediment size, and a time-dependent description of the tide, waves and storm surge.

Models such as SBEACH have application in highway design when one is interested in modeling the impacts of a severe storm on a beach seaward of a roadway. This is frequently the case on barrier islands that have shoreline parallel roads. The long-term, persistent erosion of the shoreline will ultimately lead to the exposure of the highway to overwash and flooding. An SBEACH analysis can provide guidance on the degree of highway vulnerability and the potential success of mitigation activities. These erosion control alternatives might include dune construction and beach nourishment. In both cases, an SBEACH analysis will yield information on the potential reduction in highway exposure as a result of the activity.

18.5 DESIGN WAVES

For determination of design wave, consult a coastal engineer. The following material is intended to provide an overview.

18.5.1 Linear Waves

Wind waves are created by wind blowing over the surface of the water. A schematic of a simple sinusoidal, progressive wave is given in Figure 18-11. In this Figure, the wavelength, L, and the wave height, H, are directly related to the depth of water, d. The relationship for wavelength is:

$$L = \frac{g}{2\pi} T^2 \tanh \frac{2\pi d}{L} \qquad (18.4)$$

in which T is the wave period. This Equation must be solved by iteration, although a simple approximation results from:

$$L = g \frac{T^2}{2\pi} \sqrt{\tanh \frac{4\pi^2}{gT^2} (d)} \qquad (18.5)$$

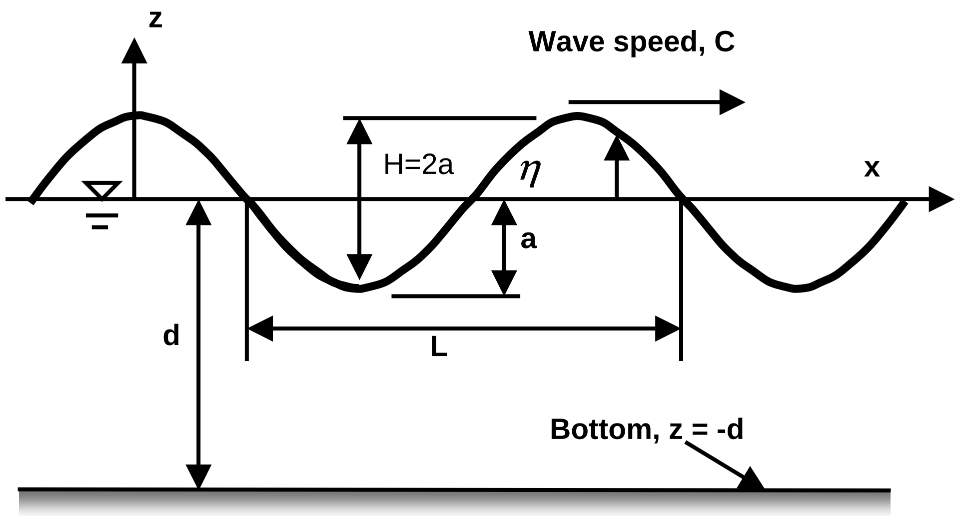


FIGURE 18-11 — Definition of Terms for a Sinusoidal, Progressive Wave (Reference *(22)*)

The speed of the propagating wave, C, is determined from L/T, where T is the wave period or time it takes for the crest to move to the position of the previous crest. The water surface variation above and below the still water level is given by:

$$\eta = H\cos\left(\frac{2\pi x}{L} - \frac{2\pi t}{T}\right) \tag{18.6}$$

where x is the horizontal distance and t is time. The waves travel freely through deep water until they become affected by changes in bottom conditions. In shallow water, refraction, diffraction and shoaling affect the wave height and length but not the wave period. The effects of wave transformation on wave height, H, are described in Section 18.6.2.3. The transformation occurs as the waves move from deep water into transitional depths and then into shallow water. The definition of deep transitional and shallow water is relative. In particular, deep water is all depth greater than one-half the wave length. Shallow water is frequently defined as d/L is less than twenty.

The pattern of waves, or wave train, on any body of water exposed to winds generally contains waves of many periods. Typical records from a recording gage indicate that heights and periods of real waves are not constant, although this is frequently assumed in the application of wave theory. The corresponding wavelength and direction of propagation are also variable and can be represented by many wavelengths. For short-crested seas, the waves appear to radiate rather than travel in a single direction. For real waves, the surface profile for waves near breaking in shallow water or for very steep waves in any water depth is distorted, with high narrow crests and broad flat troughs. Because real ocean waves are so complex, some idealization is required. This frequently leads to collapsing the irregular wave field into a harmonic wave field that is only characteristic of the actual wave conditions in a statistical sense.

Even for the simplest of cases, the estimation of wave conditions caused by meteorological conditions is complex. Elaborate numerical models requiring the use of a computer are available, but simplified techniques may be used to predict acceptable wind wave heights for the design of highway protection facilities along the shores of the ocean, embayments and inland lakes and reservoirs.

For a particular project, it is important to develop design wave conditions. Prediction of the design wave conditions is not always based upon actual measured wave conditions. Therefore, other techniques such as hindcasting are required to develop the design conditions. Wave prediction is called hindcasting when based on past meteorological conditions and forecasting when based on predicted conditions. The same meteorological conditions are used for hindcasting and forecasting. The only difference is the source of meteorological data. See Reference *(22)*, Chapter 3, for more complete information on the theory of wave generation and prediction techniques. A more complete treatment is given in Dean and Dalrymple *(8)*. In many locations, the primary wave conditions may be caused by the presence of boat or ship wakes. The prediction of wave heights from boat-generated waves is most effectively estimated from observations. However, there are some advanced tools available for estimating the characteristics of the boat wake.

18.5.2 <u>Significant Wave Height and Period</u>

A given wave train contains individual waves of varying heights and periods as shown in Figure 18-12. To represent this irregular wave field, a statistical estimation of the wave conditions are made based upon a commonly reported indicator – significant wave height. The significant wave height, H_s, is defined as the average height of the highest one-third of all the waves in a wave train (Reference *(8)*). H_s is the design wave height normally used for flexible revetments.

Other design wave heights can also be designated, such as H_{10} and H_1. If it is assumed that the wave heights follow a Rayleigh probability distribution, then other related parameters can be determined easily. The H_{10} design wave is the average of the highest 10% of all waves, and the H_1 design wave is the average of the highest 1% of all waves. The relationship of H_{10} and H_1 to H_s can be approximated as follows:

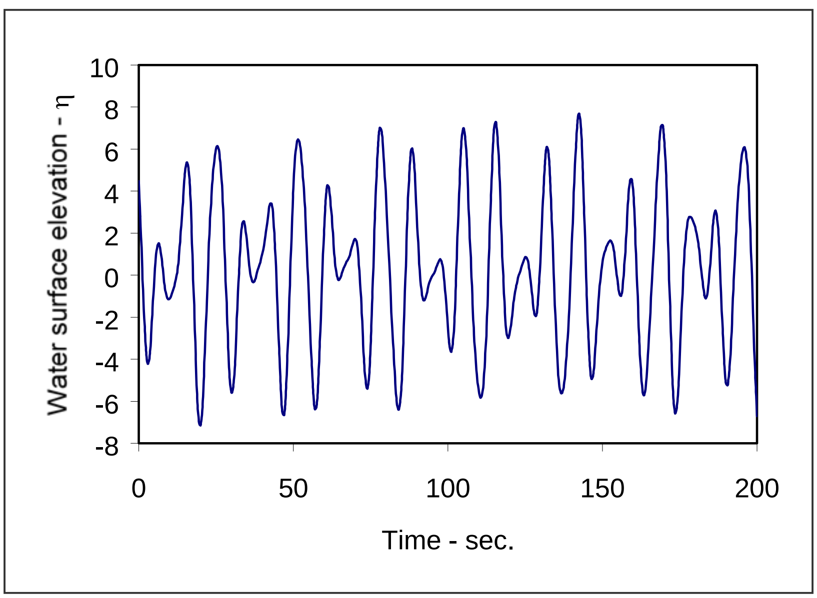


FIGURE 18-12 — Typical Irregular Wave Height History

$$H_{10} = 127H_s,\ \text{and}\ H_1 = 167H_s \qquad (18.7)$$

Alternatively, similar relationships can be developed based upon the approach of a spectral analysis of the water surface elevation. In this case, the wave heights are based upon the H_{mo}, which is a representative wave height developed from the wave spectrum. The subscript “mo” refers to the derivation of the 0^{th} moment of the wave spectrum from which the H_{mo} is computed. For most practical cases, it can be assumed that $H_{mo} = 1.1H_s$.

The above description is for a given storm or wave condition. The definition of the final design storm condition will be developed based on a series of actual annual storm events for which the maximum significant wave height for each can be determined. Then, an extreme probability analysis, such as a Gumbel or lognormal distribution, should be employed to develop the 20-, 50- or 100-yr design storm characteristics. Economics and risk of catastrophic failure are the primary considerations in designating the design wave height. A more detailed description can be found in Reference *(22)*.

18.6 METHODS FOR ESTIMATING WAVE CONDITIONS

18.6.1 Introduction

Wave height estimates for a given storm are based on wave characteristics that may be derived from an analysis of the following data:

- wave gauge records,
- visual observations,
- published wave hindcasts,
- wave forecasts, or
- maximum breaking wave at the site.

In many situations, wave gauge data are not available for a sufficiently long period from which to develop meaningful statistics. The data are useful in collaborating with hindcast data and wave transformation studies as may be required. Visual observations are typically not very accurate either from the bridge of a ship or from land. Moreover, these too are of very limited duration. Thus, much of the design storm or wave information will come from hindcast techniques.

It should be noted that deepwater ocean wave characteristics derived from offshore data analysis may also need to be transformed to the project site using refraction and diffraction techniques described in Reference *(22)* and as described in Section 18.6.2.3.

18.6.2 Predicting Wind-Generated Waves

To develop characteristics of a storm using hindcast techniques, it is important to assemble the meteorological parameters that affect the growth of the wave field. The height of wind-generated waves is a function of:

- fetch length over which the wind blows,

- windspeed,
- duration of the wind, and
- the depth of water.

Each of these meteorological parameters should be obtainable from NWS or its archives. There are several empirical models to predict the size of storm waves including the methods presented by the USACE (1992) in the EM1102-2-1502[1].

18.6.2.1 Hindcasting

Hindcasting is the use of meteorological information to infer what the waves were likely to have been under a certain storm condition. This is typically used where there are no wave gauges in place to measure the waves. Wave hindcast information, based on historical weather records and observations, is available from the USACE Waterway Experiment Station (WES) in Vicksburg, Mississippi (Reference *(5)*). USACE has conducted a *Wave Information Study* that has produced a forty-year hindcast for all shorelines on the Atlantic and Gulf Coasts and the Great Lakes. They are also working on similar results for the Pacific Ocean shorelines. The hindcast data include wave conditions at three-hour intervals over a period of forty years, along with statistical analyses of the results. These hindcasts are available at every quarter degree along the coastline. In many cases, the actual site is in 33 ft of water, and the waves must be transformed to the shoreline. These data are available from the following web site: http://bigfoot.wes.army.mil/c850.html. The procedures used in hindcasting are similar to those described below for forecasting.

18.6.2.2 Forecasting

Simplified wind wave prediction techniques may be used to establish probable wave conditions for the design of highway protection on bays, lakes and other inland bodies of water. Wind data for use in determining design wind velocities and duration is usually available from nearby weather stations, airports and major dams and reservoirs. The following assumptions pertain to these simplified methods:

- The fetch is short, 75 miles or less.
- The wind is uniform and constant over the fetch.

It should be recognized that these conditions are rarely met, and wind fields are not usually predicted as accurately as the observed wind fields. The designer should therefore not assume that the results are more accurate than warranted by the accuracy of the input and the simplicity of the method. Good, unbiased estimates of all wind-generated wave parameters should be sought and the cumulative results conservatively interpreted. The individual input parameter should not each be estimated conservatively, because this may bias the result.

The applicability of a wave forecasting method depends on the available wind data, water depth and overland topography. Water depth affects wave generation and, for a given set of wind and fetch conditions, wave heights will be smaller and wave periods shorter if the wave generation occurs in transitional or shallow water rather than in deep water. The height of wind-generated waves may also be fetch-limited or duration-limited. Selection of an appropriate design wave

[1] USACE now maintains most Engineering Manuals (EMs) on the following web site: http://www.usace.army.mil/inet/usace-docs/eng-manuals/cecw.htm.

may require a maximization procedure considering depth of water, wind direction, wind duration, windspeed and fetch length.

For deep-water wave prediction, the procedure is based on the Joint North Sea Wave Analysis Program (JONSWAP) wave spectrum parameterization. The wave spectrum is a representation of the energy in the irregular wave field that is contained in the waves of each frequency or period that exists in the wave field. The JONSWAP spectrum is based upon fetch-limited conditions and was developed based on a large field measurement program in the North Sea. The details of the JONSWAP and other parametric representations of the wave spectrum are given in Reference *(8)*. For a given wind speed and fetch length, the wave spectrum peak frequency and the spectrum can be calculated. This procedure depends on the determination of the adjusted wind speed, W_A, which is related to the measured wind speed, W, as:

$$W_A = 2.33W^{1.23} \tag{18.8}$$

in which both wind speeds are in feet per second. The adjusted windspeed is used in the following Equations to determine H_{mo} and T_p:

$$\frac{gH_{mo}}{W_A^2} = 0.0016\left(\frac{gF}{W_A^2}\right)^{0.5} \tag{18.9}$$

$$\frac{gT_p}{W_A} = 0.286\left(\frac{gF}{W_A^2}\right)^{0.33} \tag{18.10}$$

in which F is the fetch length in consistent units with gravity, g, and the adjusted wind speed, W_A, and T_P is the peak period from the wave spectrum, which is often taken as the period of the significant wave, T_S.

In Equations 18.9 and 18.10, it is assumed that the condition is fetch limited. This would be the case in most protected bodies of water and for rapidly moving storms. However, in many cases, the duration may be the limiting parameter. If the actual duration, t, is less than the minimum duration, t_d, calculated from the following Equation:

$$\frac{gt_d}{W_A} = 68.8\left(\frac{gF}{W_A^2}\right)^{0.66} \tag{18.11}$$

then, the wind is termed duration limited and the calculation should be repeated in Equations 18.9 and 18.10 using the fetch, F, obtained using the actual wind duration, t, in Equation 18.11. In the above equation, t_d is the time required for the wind to develop the waves to their highest potential.

There is no simple technique for forecasting of wind-generated waves for relatively shallow water. Several numerical models using advanced wind-wave interaction phenomena are being tested. They are difficult to employ on typical projects but may be warranted on complex or very large and costly projects. Alternatively, the method proposed for predicting shallow-water waves presented in Reference *(22)* is recommended. This approach is based upon the following Equations to obtain the significant wave height and period:

$$\frac{gH_s}{W_A^2} = 0.283\tanh\left[0.530\left(\frac{gd}{W_A^2}\right)^{0.75}\right]\tanh\left[\frac{0.00565\left(\frac{gF}{W_A^2}\right)^{0.5}}{\tanh\left[0.530\left(\frac{gd}{W_A^2}\right)^{0.75}\right]}\right]$$

(18.12)

$$\frac{gT_s}{W_A^2} = 7.54\tanh\left[0.833\left(\frac{gd}{W_A^2}\right)^{0.375}\right]\tanh\left[\frac{0.0379\left(\frac{gF}{W_A^2}\right)^{0.333}}{\tanh\left[0.833\left(\frac{gd}{W_A^2}\right)^{0.375}\right]}\right] \qquad (18.13)$$

in which the terms are as given previously. In most cases, the T_p can be considered the same as T_s. USACE (1984) provides graphs for solution of the above Equations. An example of the solution for a water depth of 15 ft is given in Figure 18-13. Additional figures are given for the wave forecast conditions at other depths in Reference *(22)*.

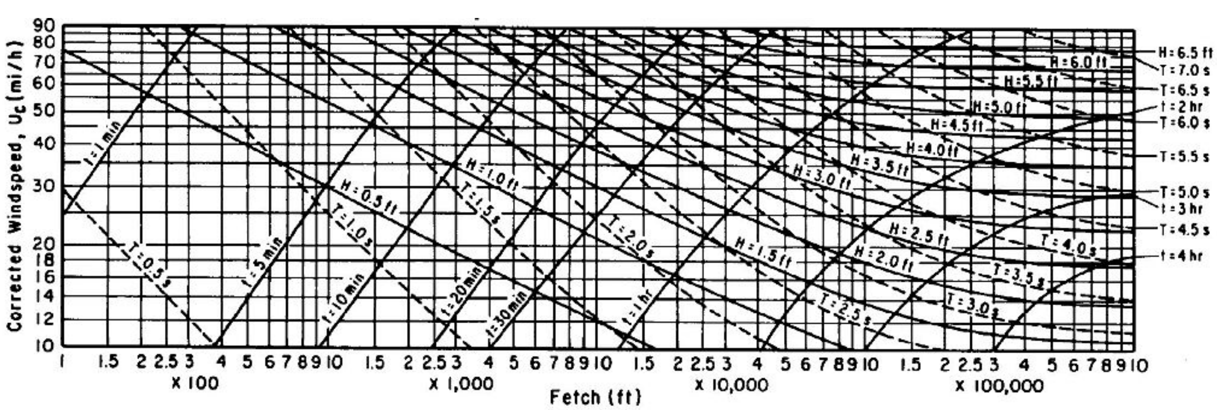
An alternative procedure to predict wave parameters is to use the ACES software that is available from USACE (Reference *(14)*). The software is available through the following web site at the Coastal & Hydraulics Laboratory of USACE: http://chl.wes.army.mil/software/cedas/gengr/acesdoc.htp. It performs the calculations from the equations provided for the deep- and shallow-water conditions. Below is an outline of a wave prediction procedure that can be used for either the methods presented above or the ACES computer software.

To estimate windspeed, the following information is needed:

- actual wind records from the site,
- general wind statistics, and
- best alternative source of wind information.

Using the method presented in Reference *(22)*, the site maximization procedure consists of the following Steps:

- Adjust wind information to 33 ft above water surface.
- Determine fetch limitations.
- Adjust wind information for over-water conditions.
- Develop and plot a windspeed-duration curve.



Note: U_C is the adjusted wind speed.

FIGURE 18-13 — Example of Significant Wave Height and Period as a Function of Windspeed, Fetch Length and Wind Duration for a Depth of 15 Ft

- When applicable, develop and plot a windspeed-duration curve for limited fetch.
- Select design wind.
- Forecast wave characteristics from significant wave prediction curves, equations or ACES.
- Determine if deepwater or shallow water conditions are present.
- For shallow water conditions, forecast the shallow water significant wave height and period (Equations 18.12 and 18.13).
- For deepwater conditions, refract and shoal the deepwater wave to the project site, if needed.
- Compute wave setup and runup.

Example Problem: Wave Forecasting

This Example makes use of the ACES program referenced above. Consider the following conditions for the forecast of wind-generated waves in shallow water:

Elevation of Observed Wind	30.0 ft
Observed Wind Speed	25 mph
Air-Sea Temperature Difference	0.0 deg
Duration of Observed Wind	3 hr
Latitude of Observation	30 degree
Average Depth of Fetch	25 ft
Length of Wind Fetch	20 nautical mi

Using the “Wind Adjustment and Wave Growth” ACES program, for shallow water conditions, these inputs result in the following wave forecast:

Wave Height = 3.62 ft
Wave Period = 4.02 sec

18.6.2.3 Wave Transformation

As waves approach the shoreline, refraction, shoaling and diffraction will affect them. Once the waves encounter a structure on the shoreline, they will be reflected back towards deeper water. However, the wave period will remain constant. The refraction is a process that occurs because of the change in wave speed as it travels over changing water depths. Because the wave speed is related to the depth and the change in direction of the wave travel is related to the wave speed, the direction the wave is traveling can be calculated for simple cases. In complicated cases of varying bathymetry, the computations can be done by graphical means or by computer programs. An example of the refraction for a wave train is given in Figure 18-14. The wave begins offshore, moving over parallel bottom contours, with an angle of α_o and arrives onshore with an angle α_3. With each change in depth, there is a corresponding change in the wave speed and, therefore, a change in direction, similar to light and sound waves as they travel through a medium in which the speed of travel changes. The process of wave refraction

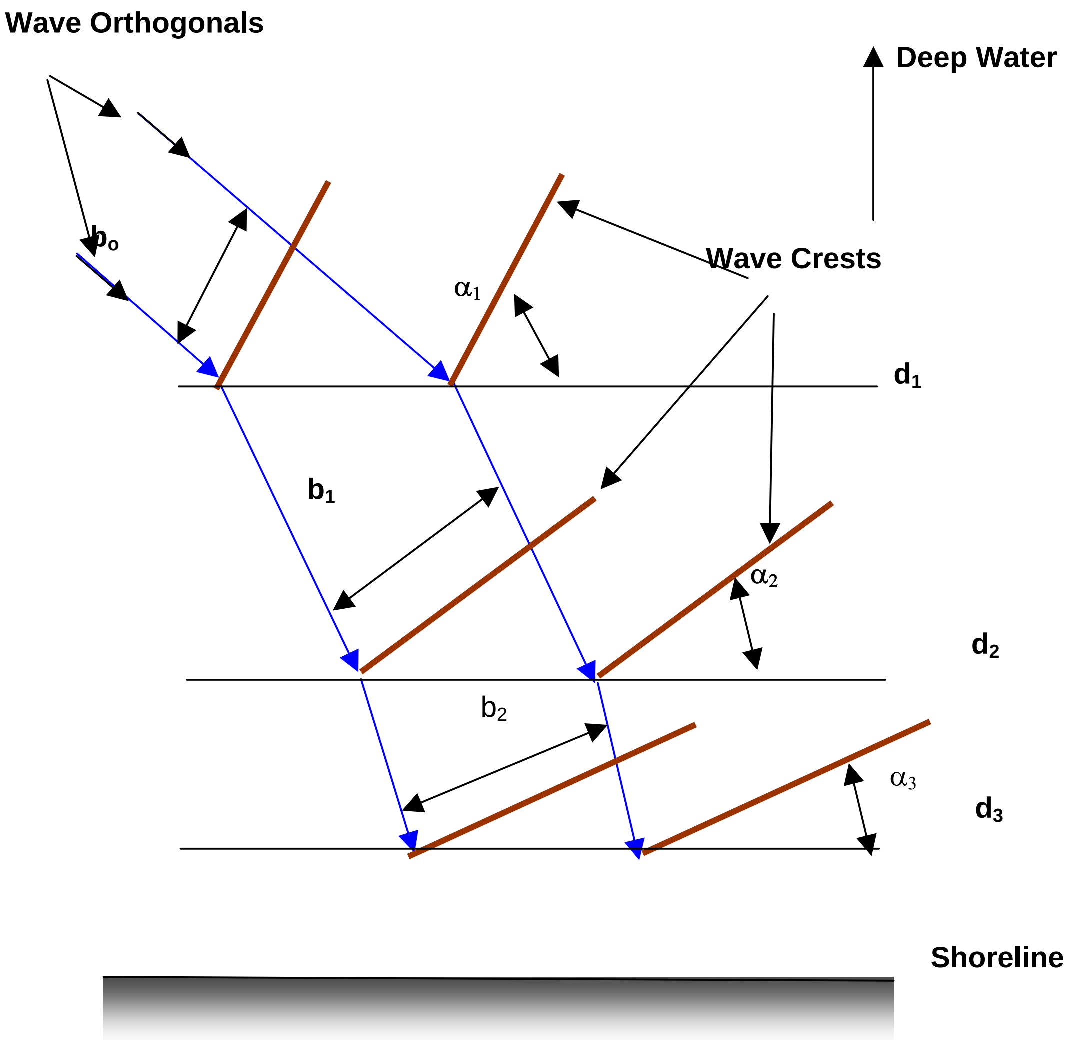


FIGURE 18-14 — Refraction Definition Showing Wave Crests and Orthogonals to the Crests as the Wave Passes from Deep through Shallow Water to the Shoreline

illustrated in Figure 18-14 is governed by Snell's Law (the same theory that governs the refraction of light). Snell's Law states that the change in wave speed is proportional to the ratio of the sines of the angles of the wave crests and the bottom contours. An example using Snell's Law and the ACES program is provided at the end of this Section.

Similarly, as the wave approaches the shoreline, the height of the wave increases and the wavelength decreases. This causes the wave to steepen and, therefore, a dual effect of refraction and shoaling occurs simultaneously. This can be quantified by looking at the power transmission past a point in the water column. For ease of computation, Figure 18-15 shows the change in the defined refraction, shoaling coefficient, K_rK_s, as a function of water depth and direction. The refraction and shoaling coefficients are defined as, respectively:

$$H(d) = K_R(d)H_o \tag{18.14}$$

$$H(d) = K_S(d)H_o \tag{18.15}$$

Both Equations represent the change in wave height with depth because the refraction and the shoaling coefficient changes with depth. When both effects are considered, the resulting wave height at water depth, d, is:

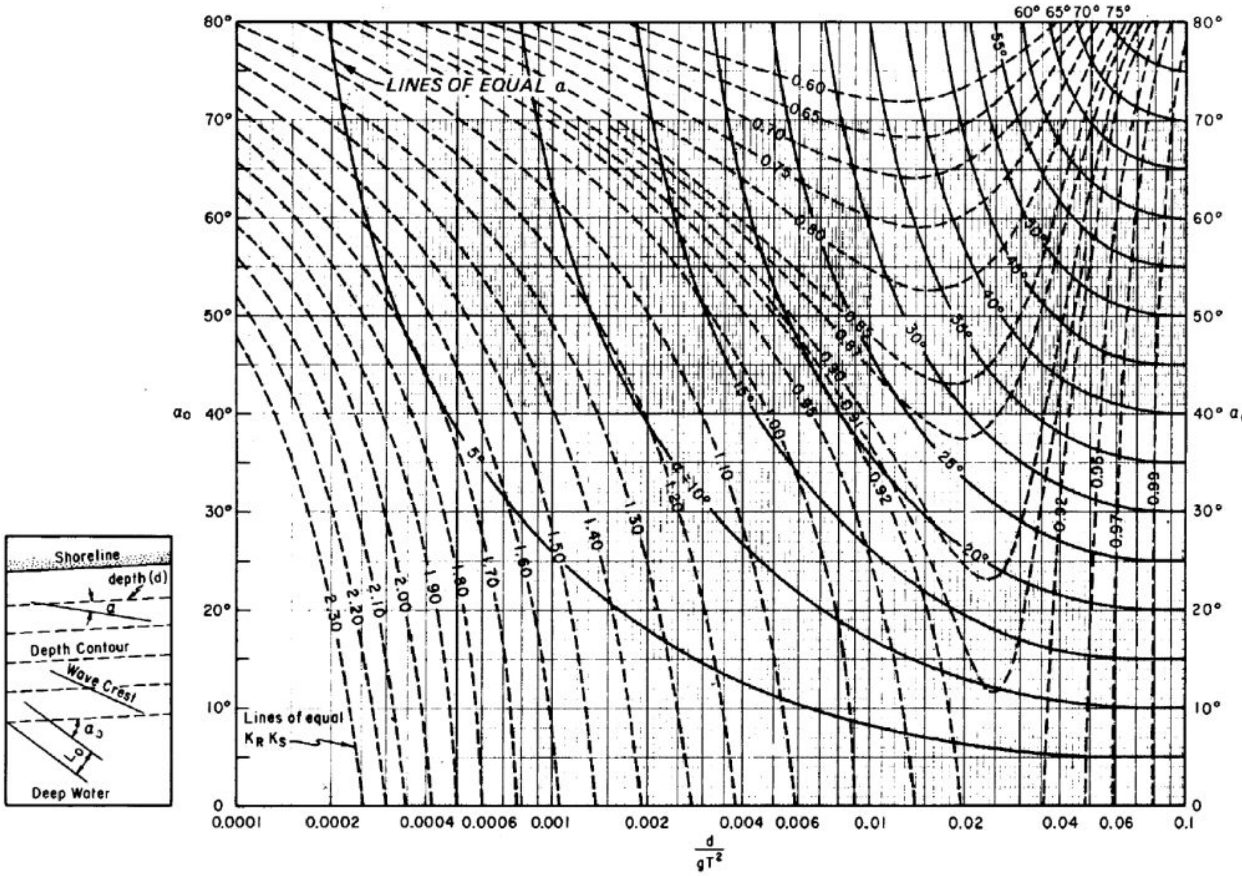


FIGURE 18-15 — Change in Wave Direction and Height Due to Refraction on Slopes with Straight Parallel Depth Contours, Including Shoaling (Reference *(8)*)

$$H(d) = K_R K_S H_o \qquad (18.16)$$

Wave diffraction occurs when waves travel behind a solid structure such as an island, causeway or breakwater. Treatment of diffraction typically proceeds using diffraction diagrams such as those provided in Reference *(22)*. Alternatively, numerical techniques can be employed such as described in Dean and Dalrymple *(8)*. The numerical techniques, while not complex, are too involved for this Chapter.

Wave reflection will occur at a solid structure with the energy being directed back to seaward. If the waves strike the structure at an angle, α, from normal to the structure, then the wave will be reflected at $-\alpha$ from the normal to the structure. For a fully reflecting structure, such as a bulkhead, and a nonbreaking wave, the reflected wave will have a height equal to the incident wave height, $H_r = H_i$, and will therefore be added to the incident wave. The result will be a wave at the structure having a height of $2H_i$. For structures that have some form of energy dissipation, such as rubble construction, or on which the waves break, the reflected wave will travel seaward with a height less than H_i.

<u>**Example Problem: Wave Refraction**</u>

This Example uses the ACES program referenced above. Consider a 10-sec wave with a height of 6 ft at a water depth of 18 ft. The angle between the wave crest and bottom contours is 6°:

$$H_1 = 6 \text{ ft}$$
$$T = 10 \text{ sec}$$
$$d_1 = 18 \text{ ft}$$
$$\alpha_1 = 6.00°$$

Using these inputs and the ACES Program "Linear Wave theory/Snells Law" results in the following results:

$$H_2 = 6.40 \text{ ft}$$
$$T = 10 \text{ sec}$$
$$d_2 = 13 \text{ ft}$$
$$\alpha_2 = 6.15°$$

This Example illustrates the process shown in Figure 18-14 whereby the refraction process results in the wave crests becoming more parallel to the depth contours as the waves move to the shoreline. The same result can be found using Figure 18-15.

18.6.2.4 Breaking Waves

Waves that approach the shoreline or a structure in shallow water are likely to break. When the waves break, they will have a wave height that is larger than was generated in deeper water as a result of shoaling unless significant refraction takes place. Thus, as the waves approach the shoreline or an embankment, they will be of a maximum size that will still be in possession of most of the deepwater energy. Wave heights derived from hindcasts or any forecasting method should be checked against the maximum breaking wave that the design stillwater level depth and near-shore bottom slope can support. The design height will be the smaller of either the maximum breaker height or the forecasted or hindcasted wave height. The relationship of the maximum height of breaker, H_b, which will expend its energy upon the shore or shore protection and depth of water at the shore protection, d_s, which the wave must pass over, are illustrated in Figure 18-16. The depth at the structure should be calculated to include the design still water level plus the wave setup. The wave setup can be computed from the breaking wave height, H_b. The wave setup is approximately $0.19H_b$. Thus, because the wave setup is dependent on the breaking height and it is dependent upon the setup plus the still water depth, the problem must be solved iteratively. Alternatively, the software ACES provides a solution to the problem directly.

18.6.2.5 Design Breaking Wave

The following Example illustrates how to use Figure 18-16 to estimate the maximum breaker wave height.

Example Problem: Breaking Design Wave

By using hindcast methods, the significant wave height, H_s, has been estimated at 3.94 ft with a 3-s period. Find the design wave height, H_b, for the slope protection if the depth of water, d_s, is 2.0 ft and the near-shore slope (m) is 1V:10H.

Solution

First, a setup of $0.19H_b$ is assumed. Assuming a breaker height of 3 ft, this gives a new depth as:

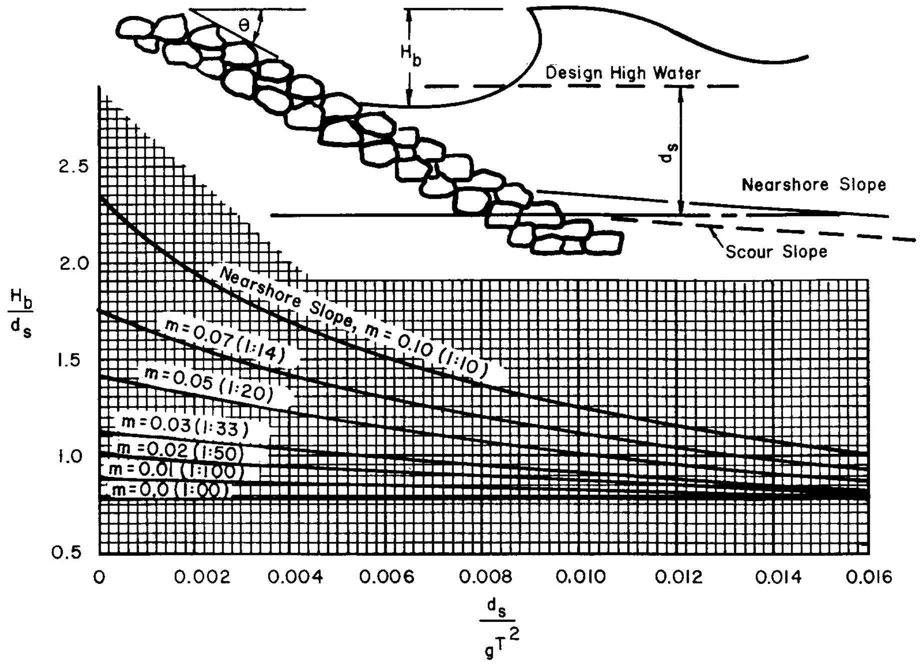


FIGURE 18-16 — Design Breaking Wave

$$d = d_s + 0.19H_b = 2.0 + (0.19)(3.0) = 2.57 \text{ ft}$$

The following dimensionless quantity is computed to use with Figure 18-16:

$$(d/gT^2) = 2.57 \text{ ft}/[(32.2 \text{ ft/s}^2)(3 \text{ s})^2] = 0.0089$$

From Figure 18-16, H_b/d_s = 1.3 and, thus, H_b = 3.9 ft. This is the expected value of the breaking wave at the toe of the structure. Because the initial value of breaker height was assumed as 3 ft, the solution is repeated using 3.9 ft, or:

$$d = d_s + 0.19H_b = 2.0 + (0.19)(3.9) = 2.7 \text{ ft}$$

Using this value of d, H_b is determined to be 3.9 ft because the variation of H_b/d_s is almost negligible. Because the maximum breaker wave height, H_b, is smaller than the significant deepwater wave height, H_s, the design wave height is 3.9 ft. This is considered to be a depth-limited condition when the breaking wave height can be used instead of the deepwater condition.

18.6.2.6 Wave Runup

An estimate of wave runup, in addition to design wave height, may also be necessary to establish the top elevation of highway slope protection as shown in Figure 18-17. Wave runup is a function of the design wave height, the wave period, bank angle and the roughness of the embankment protection material. For wave heights of 2 ft or less, runup can be estimated by using Figure 18-18 and Table 18-5. The wave runup height given on the chart is for smooth concrete pavement. The referenced wave height, H'_o, is the unrefracted deepwater wave or that normally incident wave that would have the same amplitude as the refracted wave. The

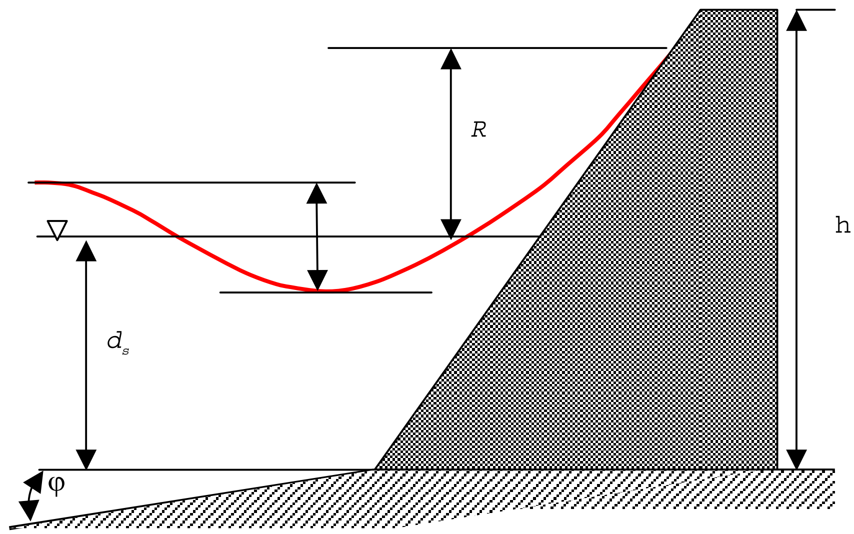


FIGURE 18-17 — Schematic of Wave Runup On Smooth Impermeable Slope

correction factors in the Table for reducing the height of runup is adequate for most highway projects. The application of more detailed procedures is rarely justified but, if needed, they are provided in Reference *(20)* and the Technical Reference for ACES (Reference *(14)*).

Example Problem: Wave Runup

Consider a deepwater wave of height 6.5 ft and period of 10 s approaching directly to the shore without refraction. The revetment has a surface of dumped stone or irregularly placed rock riprap on a slope of 1V:3H and a depth of water at the toe of 13 ft. Determine the wave runup elevation on the rubble slope.

Solution:

For the deepwater unrefracted wave height of 6.5 ft and period of 10 s, the steepness is:

$$\frac{H_o'}{gT^2} = \frac{6.5}{(32.2)(10)^2} = 0.002$$

From Figure 18-18, the value of R/H_o' is seen to be 2.9 for a slope of 1V:3H. Thus, the runup is computed as:

$$\frac{R}{H_o'} = 2.9\text{, and } R = 18.85 \text{ ft}$$

The value for runup of 18.85 ft must be corrected because the slope is composed of irregularly placed riprap. From Table 18-6, the correction is taken to be 0.5. Therefore, the final value of runup will be $R = (18.85)(0.5) = 9.43$ ft on the riprap slope.

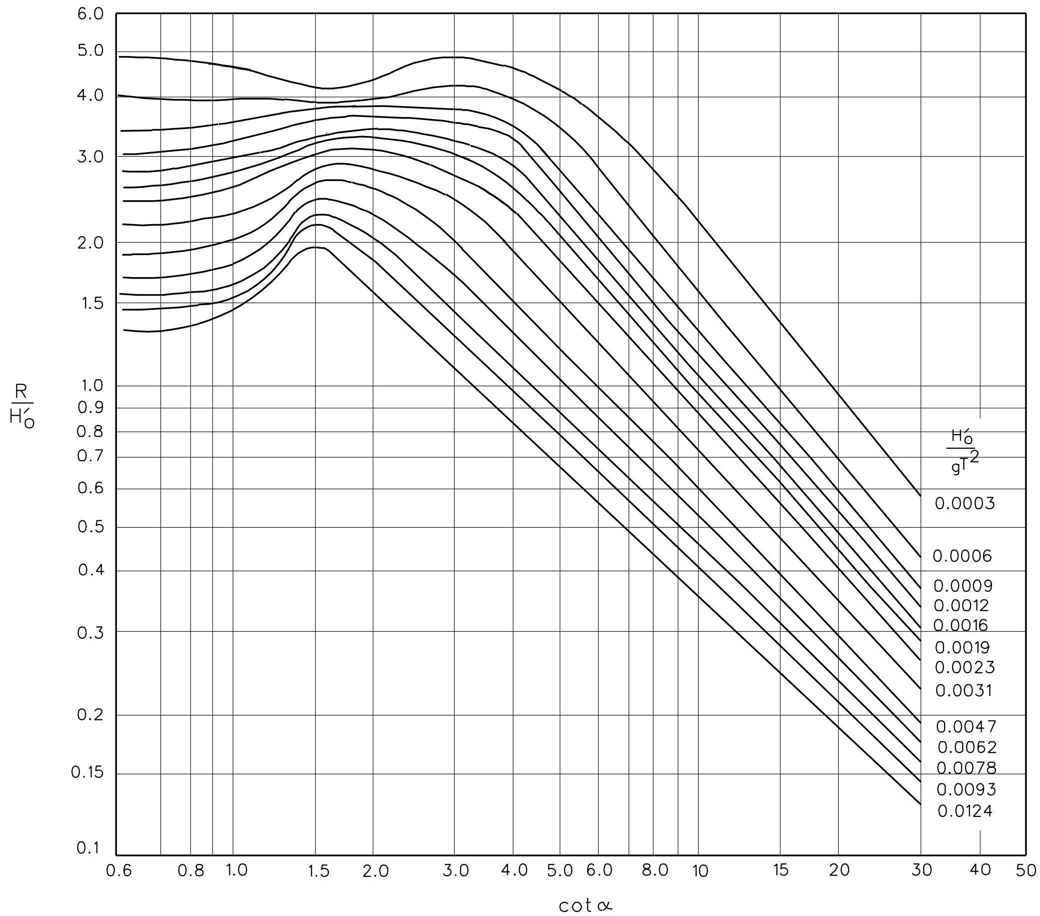


FIGURE 18-18 — Relation of Wave Runup with Wave Steepness and Bottom Slope for a Smooth Impermeable Slope

TABLE 18-6 — Correction Factors For Wave Runup

Slope Surface (Material Type)	Correction Factor
Concrete pavement	1.00
Concrete blocks (voids < 20%)	0.90
Concrete blocks (20% < voids > 40%)	0.70
Concrete blocks (40% < voids > 60%)	0.50
Grass	0.85 - 0.90
Rock riprap (angular)	0.50 - 0.60
Rock riprap (round)	0.60 - 0.65
Rock riprap (hand placed or keyed)	0.85 - 0.90
Grouted rock	0.90
Wire-enclosed rocks/gabions	0.80

18.6.3 Riprap Shore Protection

Where current velocity governs, stone size may be estimated by using the procedures in Chapter 17, Bank Protection of this *Manual*. When wave action and wind runup are significant, design of rock slope protection should proceed as described below for shore protection.

Most of the protective measures provided in Chapter 17, Bank Protection can be considered where design waves are less than 2.0 ft. For design waves greater than 2.0 ft, stone riprap usually provides the most economical and effective protection. The protection should be designed to accommodate wave runup and wind runup, if needed. Design procedures suitable for waves greater than 2.0 ft are provided below. Alternative design procedures are contained in Reference *(22)*. An alternative source is Reference *(17)*.

As discussed in the previous Section, riprap protection when used for shore protection reduces wave runup as compared to smooth types of protection. Other types of armor can be used to protect the slope, but stone is frequently the least expensive and more readily available, at least for projects for which the waves are not greater than 6 ft. Equally important in the success of the protection is the placement of the stone and the underlying filter materials. Typical sections are provided below that identify the relationship of the section to the bank of the existing shoreline. The sections also identify the toe trench that is typical with all revetments. The toe trench is used to prevent scour from occurring and undermining the revetment. Sometimes, a sheet pile wall at the toe of the revetment fulfills this function.

The proper stone or armor unit size to use for protecting a slope will be a function of the wave height, slope of the revetment, type of armor placement and specific gravity of the armor. This presumes that an adequate filter layer is included. The purpose is to design the size of the armor unit just large enough to resist the forces of the wave uprush and downrush that would cause the armor to move. Based upon numerous laboratory experiments and field verification, the "Hudson Equation" (Reference *(22)*) was developed to provide that relationship between the stone size and these parameters. The Hudson Equation is:

$$W = \frac{\gamma_s H^3}{K_D (S-1)^3 \cot\theta} \tag{18.17}$$

where:
- W = design weight of armor unit
- γ_s = unit weight of armor
- H = wave height, or typically H_{10}
- K_D = dimensionless coefficient
- S = specific gravity of armor
- θ = angle of revetment with the horizontal

Typically, W is in pounds force, H is in feet, and γ_s is in pounds force per cubic foot. The dimensionless coefficient, K_D, is based on laboratory experiments with different types of armor. Values for typical armor types are given in Table 18-7. Nomographs are provided in Reference *(17)*[2] that can be used in place of the Hudson Equation, Equation 18.17.

[2] This is currently available on the Internet at http://www.dot.ca.gov/hq/oppd/hydrology/hydroidx.htm.

TABLE 18-7 — Suggested K_D Values for Different Armor Types (Reference *(22)*)

Armor Type	Trunk	
	Breaking	Nonbreaking
Quarrystone – smooth rounded	1.2	2.4
Quarrystone – rough angular	2.0	4.0
Riprap – graded angular	2.2	2.5
Coreloc – one layer	16.0	16.0
Tetrapod	7.0	8.0
Tribar	9.0	10.0

Figure 18-19 shows an example of a revetment with a toe trench that is designed to resist scour. Riprap protection on reveted slopes are analyzed with armour sufficient to resist wave-rushing scour. Some typical design criteria for slope protection and toe trenches are:

- The toe stone is typically the same as the armour stone.
- The armour stone is placed upon a geotextile filter fabric or filter material.
- Provide revetment height freeboard for protection against the effects of wind driving the runup even higher.
- Protect the bank above the rock slope protection from splash and spray.
- Geotextile filter fabric should be tucked under the outer stones in the toe trench to prevent the fabric from unraveling from the stone.

Figure 18-19 shows an example of a revetment with a toe trench that is designed to resist scour.

In placing the toe, stone should be founded in a toe trench dug to hard rock or keyed into soft rock. If bedrock is not within reach, the toe should be carried below the depth of the scour. If the scour depth is questionable, extra thickness of rock may be placed at the toe that will adjust and provide deeper support. In determining the elevation of the scoured beach line, the designer should observe conditions during the winter season, consult records or ask persons who have knowledge of past conditions. The thickness of the protection must be sufficient to accommodate the largest stones. Except for toes on questionable foundation, as explained above, additional thickness will not compensate for undersized stones. When properly constructed, the largest stones will be on the outside and, if the wave forces displace these, additional thickness will only add slightly to the time of complete failure. As the lower portion of the slope protection is subjected to the greater forces, it will usually be economical to specify larger stones in this portion and smaller stones in the upper portion. The important factor in this economy is that a thinner section may be used for the smaller stones. If the section is tapered from bottom to top, the larger stones can be selected from a single-graded supply.

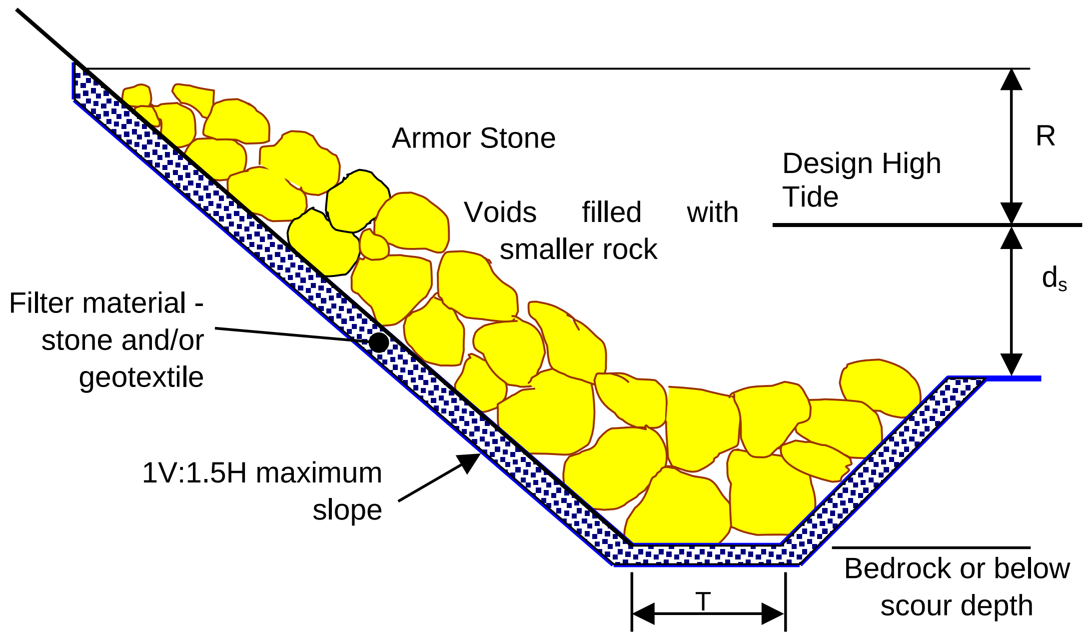


FIGURE 18-19 — Riprap Rock Shore Protection – Typical Design Configuration

18.6.4 Flooded, Vegetated Land

When waves travel across a shallow flooded area, the initial heights and periods of the waves may increase; i.e., when the wind stress exceeds the frictional stress of the ground and vegetation underlying the shallow water. The initial wave heights may decay at other times when the frictional stress exceeds the wind stress. For further discussions and example problems of estimating the growth and decay of wind waves over flooded, vegetated land, refer to Reference *(22)*.

18.6.5 ACES

The Automated Coastal Engineering System (ACES) software package can be used to solve many of the problems mentioned in this Section. ACES is a DOS-based system of computer codes and analysis packages to support the field of coastal engineers. It was developed by USACE to run on PC-type computers. The modules inside ACES are capable of simple applications, such as shown above, to very complex algorithms. The software represents many of the computations described in Reference *(22).* Typical applications are given in Table 18-8.

Information on ACES and its availability is on the web site for the Coastal & Hydraulics Laboratory at the Waterways Experiment Station of USACE. The web site for ACES is http://chl.wes.army.mil/software/cedas/gengr/acesdoc.htp. The software, although no longer free, is very useful and can be a significant help in both feasibility studies and design studies.

TABLE 18-8 — Application of ACES (Version 1.07)

Functional Area	Application
Wave Prediction	Windspeed Adjustment and Wave Growth Beta-Rayleigh Distribution Extremal Wave Height Constituent Tide Record Generation
Wave Theory	Linear Wave Theory Nonlinear Wave Theory
Wave Transformation	Linear Wave Theory Irregular Wave Transformation Combined Diffraction and Refraction
Structural Design	Revetment Design Breakwater Design using Hudson Equation Toe Protection Design Nonbreaking Wave Forces on Vertical Wall
Wave Runup and Transmission	Irregular Wave Runup on Beaches Wave Overtopping Wave Transmission Through Permeable Structures
Littoral Processes	Time-Dependent Beach and Dune Erosion Beach Nourishment Overfill Volumes
Inlet Processes	Spatially Integrated Numerical Model for Inlet Hydraulics

18.7 FLOOD PREDICTION METHODS

18.7.1 Introduction

The prediction of the flood stage and the velocities associated with the flood stage for a specific exceedence probability event are of considerable importance to the designer. The methods of prediction that are applied to coastal and lake shorelines are quite different from those used on upland rivers and streams.

18.7.2 Lake Shore Flooding

The flood stage elevation on reservoirs and sometimes on natural lakes is usually the result of inflow from upland runoff. If water stored in the reservoir is used for power generation, irrigation or low-water augmentation, or if the reservoir is used for flood control, the level of the water at the time of a flood must be anticipated from a review of operating schedules. In the absence of such data, the designer should assume a conservative approach and use a high starting lake level. Wind-generated waves will also be present in many flood instances.

A highway design should reflect consideration of flood levels, wave action and reservoir operational characteristics. However, an attempt to provide the highway facility with protection from the rare flood events normally used in the design of a reservoir would seldom provide a cost-effective design.

Reservoir routing techniques are used to predict the stillwater flood levels for most lakes and reservoirs. These levels should be increased appropriately to reflect the superimposition of waves. Very large lakes, such as the Great Lakes, are treated as coastal areas. There are also long-term water level records available for each of the Great Lakes that will be important when designing projects there. Seiching is an important problem at the Great Lakes and must be considered in the design. Lake level seiching of up to 6.5 ft is possible in some areas such as the shallow end of Lake Erie near Toledo.

Lakes have insignificant tidal variations but are subject to seasonal and annual hydrologic changes in water level and to water level changes caused by wind setup, barometric pressure variations and seiches. Additionally, some lakes are subject to occasional water level changes by regulatory control works.

18.8 BRIDGE AND CULVERT DESIGN TECHNIQUES

18.8.1 Introduction

The design of coastal or tidally influenced waterway bridge openings is typically more complicated than similar designs on riverine systems. The natural system is difficult to quantify because it is typically spatially variant in two horizontal directions and it is unsteady flow. Coastal waterways are subject to storm surges and astronomical tides that play an important role in the dynamic hydraulic behavior of the system. The collection of adequate data to represent the actual condition also adds to the complexity of the problem. Data such as flows and storm surge description may be very difficult to estimate.

In many cases, the bridge hydraulic opening is designed to extend across the normal open water section. This may be an appropriate design from an economic standpoint, because the total cost of a larger bridge approximates the cost of a smaller bridge considering approach embankments and abutment protection measures. This design is also desirable from an environmental perspective because it results in minimal environmental impacts. In most designs, the extent of detail in the analysis must be commensurate with the project size or potential environmental impacts. However, analytical evaluation of the opening is often required and is necessary when a full crossing cannot be considered or when the existing structure and channel exhibits hydraulic problems. The complexity of these analyses lends themselves to computer modeling.

18.8.2 Storm Surge Determination

As noted in Section 18.7, it is frequently acceptable to determine the frequency-indexed storm surge height along the open coast using the results from FEMA flood maps. In some locations, NOAA has also produced storm surge height information along the coast. Their results can also be used in the bays and estuaries, but the major drawback is the lack of dynamic information. For example, most bridge scour design and analysis depends upon defining the time history of the storm surge and the tides. This is very important to determine the velocity field and history that will control scour computations.

Reference *(4)* provides three techniques for determining the storm surge hydrograph at the coast. The techniques basically produce the boundary condition that would be used in one of the computer techniques described in the Section for determining the surge plus tide elevation and velocity field throughout the estuary for the duration of the storm.

The recommended method for most cases is the single design hydrograph (SDH). The SDH is based upon measurements and hindcast studies of the historical storm surges associated with tropical storms that have entered the East and Gulf Coasts. The SDH is somewhat similar to the method proposed by Cialone, et al. *(7)*. Considering the pressure distribution given in Equation 18.2, the surge plus tide height history is approximated by:

$$S_{tot}(t) = S_p\left(1 - e^{-|D/t|}\right) + H_t(t) \tag{18.18}$$

where:
- D = R/f = storm duration
- R = radius to maximum winds
- f = forward speed
- t = time
- H_t = height of astronomic tide
- S_p = known storm surge height

To obtain the value of S_p to use, one can use the published results of the SURGE, SLOSH or ADCIRC results. In Florida, special studies have been conducted by the Department of Environmental Protection to provide storm surge estimates for a 100-yr event. The SURGE and SLOSH estimates include both surge and tide components. The surge estimates of ADCIRC are based upon historic storms but do not include any tide component. So, care should be exercised in choosing the appropriate source for the value of the maximum surge S_p. These data are not available for the Pacific and Great Lakes coastlines. Another consideration is choosing the correct tide conditions to add to the surge hydrograph. Figure 18-20 shows the base surge hydrograph and the effects of combining the surge hydrograph such that the peak occurs at the same time as low tide, high tide, midrising tide and midfalling tide. As can be seen, the results are clearly different for each case, and it can be correctly assumed that the effects up the estuary or tidal river will be different for each case. Note in the Figure that the effect of the time of the high tide has a significant effect on the total storm surge.

18.8.3 <u>Computer Modeling</u>

Existing models cover a wide range from simple analytical solutions to computationally intensive, numerical models. Some models deal only with flows through inlets, while others describe general one-dimensional or two-dimensional flow in coastal areas. A higher level includes hurricane or other storm behavior and predicts the resulting storm surges.

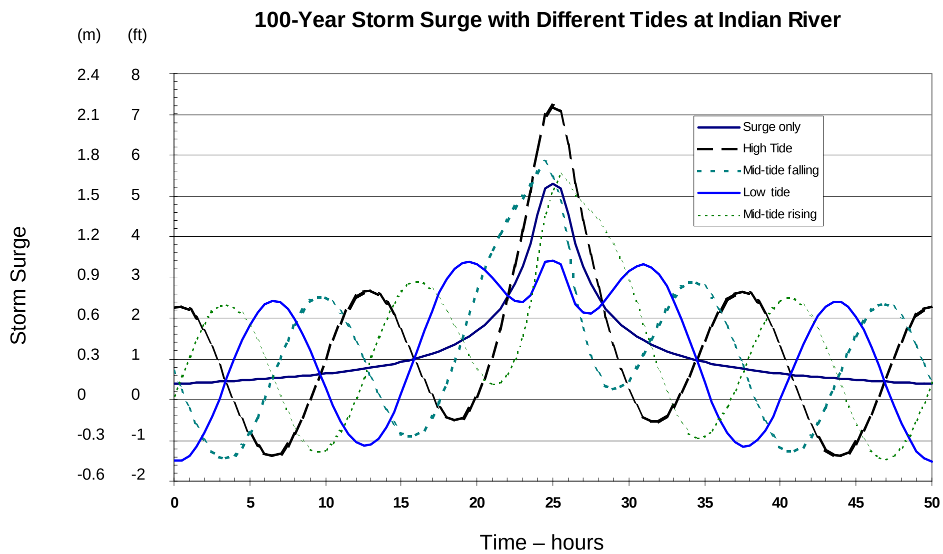


FIGURE 18-20 — Demonstration of the Effect of Combining the Storm Surge with the Astronomic Tides at Different Phases for Indian River, Delaware (Reference *(3)*)

One-dimensional models are the most commonly used models because they demand less data and computer time than the more comprehensive models. Most analyses for tidal streams are conducted with steady-state models where the tidal effects are not simulated. This may be an adequate approach if the crossing is located inland from the mouth where the tidal effects are minor. Computer modeling for steady-state hydraulics is generally preformed with the USACE's HEC-RAS, USDA/NRCS WSP-2 or the USGS/FHWA WSPRO (HY-7) (Reference *(2)*). In Phase 1 of the Pooled-Fund study lead by the South Carolina Department of Transportation (Ayres Associates 1994), UNET (Barkau 1996) and FESWMS-2DH[3] (Reference *(10)*) were recommended for dynamic tidal hydraulic modeling. UNET is a one-dimensional model, and FESWMS-2DH is a two-dimensional model. Although other one- and two-dimensional models are also applicable for tidal hydraulic modeling, the recommended models incorporate bridge, culvert and road overtopping hydraulics. Therefore, these models were deemed most applicable for tidal bridge hydraulic and scour evaluations. Detailed directions for model development for flow in tidal waterways is given by Ayres *(4)*. This includes flow charts for development, calibration and model use and the use of the two-dimensional model within the modeling shell, SMS. This will be discussed in more detail in Section 18.8.3.3.

18.8.3.1 One-Dimensional Models

UNET is a powerful, unsteady-flow model that computes flow though a bifurcating or branching network of channels. UNET also includes storage areas and a wide variety of hydraulic structures. These features make this model useful in tidal and unsteady riverine applications where bridge hydraulics are an important component. Under agreement with Dr. Barkau, developer of the model, the USACE Hydrologic Engineering Center (HEC) maintains, distributes and provides training for UNET. Version 3.0 of HEC-RAS contains the one-dimensional model UNET developed by Barkau (1996) *(12)*. The input to the program consists of channel geometry as a series of cross sections comprising a channel reach, flow-resistance parameters, ineffective flow areas, structures located on or along the channels, storage areas located at the ends or adjacent to the channels, information on how the channel reaches and how storage areas are connected, and boundary conditions. Structures include bridges, culverts, roadway embankments, spillways, navigational dams and closed conduits.

Dynamic models perform hydraulic computations for channels, overbanks, bridges and culverts and include potentially filled or inundated bay, estuary and floodplain areas. Therefore, dynamic models yield the most accurate hydraulic analysis for flooding and scour computations and countermeasure design. The governing equations used in these models are the full dynamic equations for conservation of mass and momentum. One-dimensional modeling is applicable for estuaries with well-defined channels and for bays with single or multiple inlets. Where bays are crossed by numerous causeways, especially causeways with multiple bridge openings, two-dimensional modeling is recommended. Estuaries with multiple-branched channels and nearly well-defined channel flow can be modeled with one-dimensional network models (such as HEC-RAS), but the complexity often warrants the use of two-dimensional models. When it is necessary to employ dynamic modeling, it must be recognized that the development time will be significantly longer than for the one-dimensional model. However, it does provide a tool to develop complex and dynamic flow patterns.

---

3 The newest version of FESWMS-2DH is Version 3 *(10)*. In Version 3, FESWMS-2DH is renamed Flo2DH. Future versions of the User's Manual will be referenced on the web site of FEMA, Bridge Division, http://www.fhwa.dot.gov/bridge/hyd.htm.

18.8.3.2 Two-Dimensional Models

The computer program Finite Element Surface Water Modeling System: Two-Dimensional Flow in a Horizontal Plane (FESWMS-2DH) is maintained by FHWA. As indicated by its name, FESWMS-2DH uses a finite-element numerical method to solve the equations that describe the two-dimensional flow of shallow surface water. The governing equations include the conservation of mass, the conservation of momentum in two directions and boundary condition equations. The program solves for the flow depth and the x- and y-velocity components at discrete points, called "nodes," throughout the network.

The input to the program includes information on the network and the boundary conditions. The network is the discretized spatial description of the system being modeled and consists of elements and nodes. Elements are 3- or 4-sided cells that may be irregular in shape and size. Each element has an associated material type that contains the values of resistance parameters. Nodes are the points at the corners and midsides of the elements. Each 4-sided element also has a node at its center. Every node has a bed elevation assigned to it to describe the bathymetry of the system.

Another effective two-dimensional model that can be used is RMA2 (AYERS, 1994). Like FESWMS, this model is a two-dimensional depth, averaged finite-element hydrodynamic numerical model. It computes water surface elevations and horizontal velocity components for subcritical, free-surface flow in two-dimensional flow fields. RMA2 computes a finite-element solution of the Reynolds form of the Navier-Stokes equations for turbulent flows. Friction is calculated with the Manning's or Chezy Equation, and eddy viscosity coefficients are used to define turbulence characteristics. The model is used to compute dynamic response to tidal, storm surge, winds and steam inflow hydrographs. Similar to FESWMS, the output from the model is water surface elevation and velocity throughout the domain.

18.8.3.3 Surface-Water Modeling System

Surface-water Modeling System (SMS) is a pre- and post-processor that can be used with both FESWMS and RMA2. The SMS software is a comprehensive graphical user environment for two- and three-dimensional numerical modeling. The Environmental Modeling Research Laboratory at Brigham Young University developed it in cooperation with the USACE Waterways Experiment Station, now ERDC (Engineering Research and Development Command) and FHWA. The finite-element mesh or cross section entities, plus the associated boundary conditions necessary for analysis, are created within SMS and then saved to model-specific files. These files are used as input to the hydrodynamic, wave mechanic, contaminant migration and sediment-transport analysis engines of the models supported by SMS, such as RMA2 and FESWMS. The numerical models create solution files that contain the water surface elevations, velocities, contaminant concentrations, sediment concentrations or other functional data at each node, cell or section. Plots and animations can then be easily created within SMS to view the simulation results. SMS can also be used as a pre- and post-processor for other finite-element or finite-difference programs if the programs can read and write files in a supported format. SMS is capable of constructing large, complex meshes (up to hundreds of thousands of elements) of arbitrary shape. An example of part of a mesh generated within SMS is given in Figure 18-21. This shows part of the mesh for Corpus Christi Bay and part of the Gulf of Mexico.

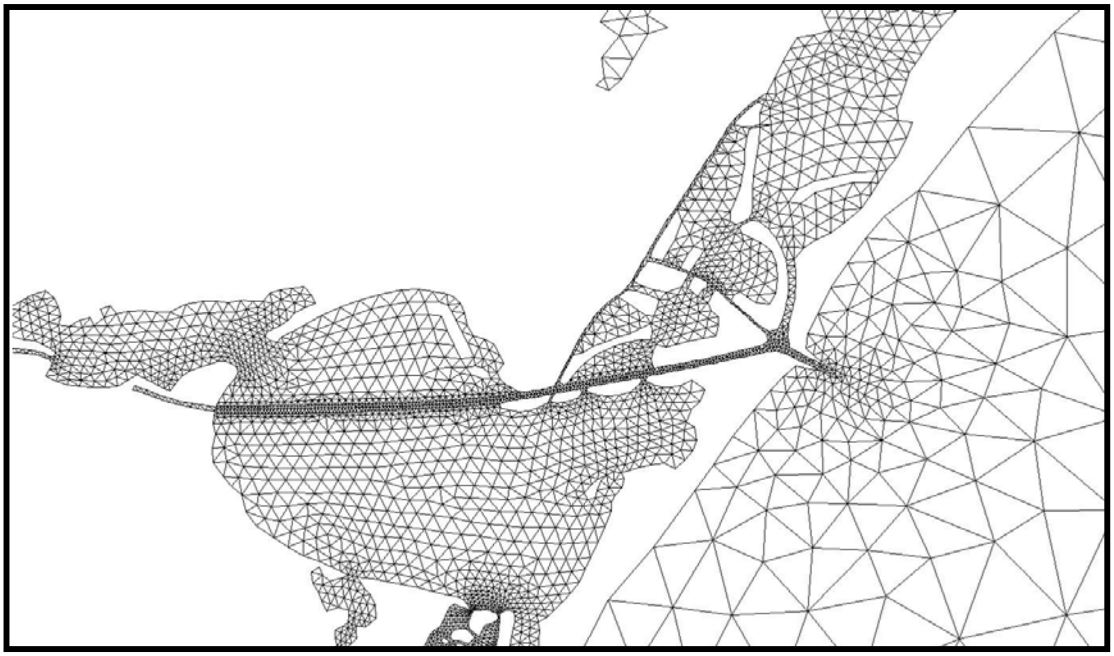


FIGURE 18-21 — Example of Finite Element Grid for Corpus Christi Bay Using Triangular Elements

SMS can also be used as a pre- and post-processor for many other surface water modeling tools for analysis and design. Supported models include the USACE-WES supported TABS-MD (GFGEN, RMA4, RMA10, SED2D-WES) in addition to the RMA2 mentioned above. It also supports ADCIRC, CGWAVE, STWAVE and HIVEL2D. Comprehensive interfaces have also been developed for facilitating the use of the FHWA-commissioned analysis packages FLO2DH (formerly FESWMS) and WSPRO. SMS also includes a generic interface, which can be used to support models that have not been officially incorporated into the interface. The SMS pre- and post-processor includes two-dimensional finite element, two-dimensional finite difference, three-dimensional finite element and one-dimensional backwater modeling tools.

18.9 REFERENCES

(1) AASHTO, *Highway Drainage Guidelines,* Chapter 11, "Highways Along Coastal Zones and Lakeshores," Task Force on Hydrology and Hydraulics, 2003.

(2) Arneson, L.A. and Shearman, J.O., *User's Manual For WSPRO — A Computer Model for Water-Surface Profile Computations*, Publication No. FHWA-SA-98-080, Federal Highway Administration, Washington, DC, 1998.

(3) Ayres Associates, "Development of Hydraulic Computer Models to Analyze Tidal Coastal Stream Hydraulic Conditions at Highway Structures," Final Report, Phase I HPR552, South Carolina Department of Transportation, Columbia, SC, 1994.

(4) Ayres Associates, "Development of Hydraulic Computer Models to Analyze Tidal Coastal Stream Hydraulic Conditions at Highway Structures," Final Report, Phase II HPR552, South Carolina Department of Transportation, Columbia, SC, 1997.

(5) Brooks, R.M. and Brandon, W.A., "Hindcast Wave Information for the US Atlantic Coast: Update 1976-1993 with Hurricanes" WIS Report 33(AD A298 660), US Army Engineer Waterways Experiment Station, Vicksburg, MS 39180-6199, 1995.

(6) Cardone, V.J., Greenwood, C.V. and Greenwood, J.A., "Unified Program for the Specification of Hurricane Boundary Layer Winds Over Surfaces of Specified Roughness," US Army Corps of Engineers, Coastal Engineering Research Center, CERC-92-1, Vicksburg, MS, 1992.

(7) Cialone, M.A., Butler, H.L. and Amein, M., "DYNLET1 Application to Federal Highway Administration Projects," Miscellaneous Paper CERC-93-6, Coastal and Hydraulics Laboratory, Waterways Experiment Station, Vicksburg, MS, 1993.

(8) Dean, Robert G. and Dalrymple, Robert A., *Coastal Processes for Engineers*, (In press), World Scientific, Singapore, 2001.

(9) Federal Emergency Management Agency, "Flood Plain Management: Ways of Estimating Wave Height in Coastal High Hazard Areas, in the Atlantic and Gulf Coast Regions," TD-3, 1981.

(10) Froehlich, D. C., "Finite Element Surface-Water Modeling System: Two-Dimensional Flow in a Horizontal Plane," FESWMS-2DH, Version 3, User's Manual, Federal Highway Administration, FHWA-RD-03-053, Turner-Fairbank Highway Research Center, McLean, VA, 1996.

(11) Harris, D. L., Meteorological Aspects of Storm Surge Generation, *Journal of the Hydraulics Division,* ASCE, No. HY7, 1958.

(12) HEC, "UNET – One-Dimensional Unsteady Flow Through a Full Network of Open Channels," User's Manual, CPD-66 Version 3.2, US Army Corps of Engineers, Hydrologic Engineering Center, Davis, CA, 1997.

(13) Landsea, C.W., "FAQ: Hurricanes, Typhoons and Tropical Cyclones," NOAA/AOML, Miami, Florida, 2000.

(14) Leenknecht, D.A., Szuwalski, A. and Sherlock, A.R., *Automated Coastal Engineering System,* Coastal and Hydraulics Laboratory, Waterways Experiment Station, Vicksburg, MS, 1992.

(15) National Academy of Sciences, “Methodology for Calculating Wave Action Effects Associated with Storm Surges,” Washington, DC, 1977.

(16) NOAA, *Our Restless Tides,* U.S. Government Printing Office, Washington, DC, 1998.

(17) Racin, James A., Hoover, Thomas P. and Crosset-Avila, Catherine, “California Bank and Shore Rock Slope Protection Design - Practioner’s Guide and Field Evaluations of Riprap Methods,” FHWA-CA-TL-95-10, Caltrans Study No. F90TL03, 3rd Edition – Internet, 2000.

(18) Reid, R.O. and Bodine, B.R., Numerical Model for Storm Surges, Galveston Bay, *Journal of the Waterways and Harbors Division,* ASCE, Vol. 94, No. WW1, 1968.

(19) US Army Corps of Engineers, “Coastal Littoral Transport,” EM1110-2-1502, Waterways Experiment Station, Vicksburg, MS, 1992.

(20) US Army Corps of Engineers, “Design Of Coastal Revetments, Seawalls, and Bulkheads,” EM1110-2-1614, Coastal Engineering Research Center, Waterways Experiment Station, Vicksburg, MS, 1995.

(21) US Army Corps of Engineers, “Engineering and Design Storm Surge Analysis,” EM1110-2-1412, Waterways Experiment Station, Vicksburg, MS, 1986.

(22) US Army Corps of Engineers, *Shore Protection Manual,* Volumes I and II, Coastal Engineering Research Center, Waterways Experiment Station, Vicksburg, MS, 1984.

(23) Westerink, J.J., Blain, C.A., Luettich, R.A. and Scheffner, N.W., “ADCIRC: An Advanced Three-Dimensional Circulation Model for Shelves, Coasts, and Estuaries. Report 2: Users Manual for ADCIRC-2DDI,” Tech. Rep DRP-92-6, US Army Corps of Engineers, Coastal Engineering Research Center, Vicksburg, MS, 1994.